INTEGRATED APPROACH TO URBAN WASTEWATER MANAGEMENT

By

SHEIKH MOHAMMAD HASAN

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To My Family

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

NOTATION

The following symbols have been adopted for use in this study:

a	= coefficient (inches per hour)
a_{ij}	= coefficient for city i in region j (inches per hour)
A	= area (acres)
AR	= annual runoff (inches per year)
AR_i	= annual runoff in city i (inches per year)
AR_j	= annual runoff in test city for region j (inches per year)
b	= coefficient (inches per hour)
b_j	= coefficient for test city in region j (inches per hour)
b_{ij}	= coefficient for city i in region j (inches per hour)
B	= runoff coefficient
B_D	= runoff coefficient after development
B_u	= runoff coefficient prior to development
BOD	= biochemical oxygen demand (mg/l)
BOD_5	= biochemical oxygen demand at 5 days (mg/l)
c	= unit cost of control ($)
c_i	= unit cost of resource i ($)
c_o	= initial concentration (mg/l)

c_t = concentration at time t (mg/l)

c_{ij} = unit cost of control i in area j ($)

c_E = unit treatment cost for excess capacity (annual collars per inch per hour

c_S = unit cost of storage (annual dollars per acre-inch)

c_T = unit cost of treatment (annual dollars per acre-inch per hour)

c(i) = characteristic function representing individual cost ($/year)

c(ij) = characteristic function representing joint cost ($/year)

c_D = unit cost of dry-weather treatment (dollars per pound)

c($\underline{S}$) = characteristic function representing cost to caolition $\underline{S}$ ($/year)

c(N) = characteristic function representing total cost ($/year)

C_i^ψ = annual control cost to city i for treatment level ψ ($/year)

C_{ij}^ψ = annual control cost to cities i and j for treatment level ψ ($/year)

CA = amortized capital costs ($/year)

CA^ψ = amortized capital costs for treatment level ψ ($/year)

CA_i^ψ = amortized capital costs to city i for treatment level ψ ($/year)

d = coefficient (inch^{-1})

d_{ij} = coefficient for city i in region j (inch^{-1})

D = annual average dry-weather flow (mgd)

$\hat{D}$ = design capacity of drv-weather plant (mgd)

D_i = annual average dry-weather flow from city i (mgd)

$\hat{D}_i$ = design capacity of dry-weather plant (mgd)(city i)

DWB = annual quantity of dry-weather BOD (pounds per acre-year)

DWF = annual quantity of dry-weather flow (inches per year)

DO = dissolved oxygen (mg/l)

e^{z_p} = annual cost of wet-weather pollution control, in eastern U. S., using primary devices ($/acre)

e^{z_s} = annual cost of wet-weather pollution control, in eastern U. S., using secondary devices ($/acre)

E = maximum weighted difference between an objective and its maximum value

f = coefficient (percent R)$^{-1}$

f_{ij} = coefficient for city i in region j (percent R)$^{-1}$

F = total annual cost ($)

$F_1(x)$ = sum of resource costs ($)

$F_2(y)$ = sum of damages ($)

$F_j(x)$ = resource costs to purpose or group j ($)

$g(X,Y)$ = production function

$g_i(x)$ = constraint set

$g_{ij}(x)$ = constraint set for purpose or group j

$g_{ij}(X,Y_j)$ = constraint set

$g(R,S,T)$ = production function relating percent control (R) to storage (S) and treatment (T)

G = conversion factor

h = coefficient (percent R)$^{-1}$

h_j = coefficient for test city in region j (percent R)$^{-1}$

h_{ij} = coefficient for city i in region j (percent R)$^{-1}$

H = imperviousness, a fraction of percent

i = area or input

I = intensity of rainfall (inches per hour)

IC_D = incremental cost of dry-weather treatment ($)

j = control option or output

J = constant

k = BOD removal rate constant, time^{-1}

K = constant

K_A = constant

l = constant

l_ψ = constant for treatment level ψ

L = pipeline distance (miles)

L_{ij} = pipeline distance from i to j (miles)

L^*_{ij} = break-even pipeline distance from i to j (miles)

LA = Lagrangian function

m = exponent (less than 1)

m_ψ = exponent for treatment level ψ (less than 1)

M = pollutant loading averaged over different land uses (lbs/year)

M_1 = pollutants removed from wet weather (lbs/year)

M^*_1 = optimal amount of pollutants removed from wet weather prior to initiating tertiary treatment (lbs/year)

M_c = pollutant loading in combined sewered areas

M_j = maximum value of objective j

M_s = pollutant loading in separate sewered areas

M_{tert} = increased dry-weather BOD removal through tertiary treatment (lbs/acre)

MB_j = marginal benefit of output j ($)

MC_i = marginal cost of i (\$)

MRS = marginal rate of substitution

MRT = marginal rate of transformation

N = nitrogen content of wastewater (mg/l)

NSC = nonseparable cost (\$/year)

OM = operation and maintenance cost (\$/year)

OM^{ψ} = operation and maintenance cost for treatment level ψ (\$/year)

OM_i^{ψ} = operation and maintenance to city i for treatment level ψ (\$/year)

P = constant

P_{ψ} = constant for treatment level ψ

P_j = unit price for output j (\$)

P = precipitation rate (inches per year)

PD = population density (persons/area)

q = exponent (less than 1)

q_{ψ} = exponent for treatment level ψ

Q_i = release for city or area i

r_1 = constant

r_2 = constant

R = percent runoff control

R_1 = percent pollutant control

$\overline{R}_1$ = maximum percent pollutant control

RO = runoff rate (cubic feet per sec)

s = number of players

$\underline{S}$ = coalition with s players

S = storage volume (mg or inches)

S^* = optimal storage volume (inches)

SC_i = separable cost to city in purpose i

t = detention time, time

t_i = unit cost of transmission for area i ($)

T = treatment rate (inches per hour)

T^* = optimal treatment rate (inches per hour)

$\underline{T}$ = coalition $\underline{T}$

T_1 = treatment rate at which isoquant is parallel to the ordinate (inches per hour)

T_2 = treatment rate at which isoquant intersects the abcissa (inches per hour)

TC = total annual cost ($/year)

TC^{ψ} = total annual cost for treatment level ψ ($/year)

TC_{sec} = total annual cost for secondary treatment ($/year)

TC_{tert} = total annual cost for tertiary treatment ($/year)

u = constant

u_{ψ} = constant for treatment level ψ

U = constant

v_{ψ} = constant for treatment level ψ

$v(i)$ = characteristic function for player i

$v(\underline{S})$ = characteristic function for coalition $\underline{S}$

$v(N)$ = characteristic function for all players

V = volume of storage required for wet-weather quantity control (acre-feet)

$\overline{V}$ = maximum allowable release (acre-feet)

V_i = wet-weather quantity control volume required by city i (acre-feet)

w = constant

w_1 = constant

w_2 = constant

$w^z p$ = cost of wet-weather control using primary device for the western U. S. (\$/acre)

$w^z s$ = cost of wet-weather control using secondary device for the western U. S. (\$/acre)

$\overline{W}$ = allowable release

x = input

x_i = ith input

$x(i)$ = cost assigned to group or purpose i

$x(ij)$ = quantity controlled by th control option in area i

$\overline{x}_{ij}$ = maximum available control by jth control option in area i

X = input vector

y = outputs

y_j = jth output

y = output vector

z = constant

z_1 = constant

z_2' = constant

Z = total annual cost (\$/acre)

Z^* = optimal total annual cost (dollars per acre)

$Z_k(X)$ = kth objective

Z_p = annual cost for primary control unit (dollars per acre)

Z_s = annual cost for secondary control unit (dollars per acre)

Z_1 = annual cost of dry-weather quality control (dollars per acre)

Z_2 = annual cost of wet-weather quality control (dollars per acre)

Z_3 = annual cost of wet-weather quantity control (dollars per acre)

Z_{12} = annual cost of dry-weather quality control and wet-weather quality control (dollars/acre)

Z_{23} = annual cost of wet-weather quality control and wet-weather quantity control (dollars/acre)

Z_{13} = annual cost of dry-weather quality control and wet-weather quantity control

Z_{123} = annual cost of dry-weather quality, wet-weather quality, and wet-weather quantity control (dollars per acre)

$\alpha(i,j)$ = pollutant load j from separate area land use i

$\beta(i,j)$ = pollutant load j from combined area land use i

β_i = allocation vehicle

γ = imputation vector

γ_i = imputation i

δ = imputation vector

δ_i = imputation i

ψ = treatment level

ϕ = treatment level

η = efficiency of treatment

π = shadow price

ϕ_i = cost sharing and/or cost allocation

λ = Lagrange multiplier

η_s = efficiency of secondary treatment

η_t = efficiency of tertiary treatment

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By

Sheikh Mohammad Hasan

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Traditionally, the problem of urban wastewater management has been viewed as the control of urban storm drainage and dry-weather sewage treatment. Each of these purposes has been accomplished independently in a least-cost manner. With the enactment of Public Law 92-500, communities within the designated urban areas are required to plan for coordinated wastewater management. A third facet of urban wastewater management, i.e., wet-weather quality control, has also become important as the nation strives to achieve the 1983 goal of "swimmable and fishable" waters. Further, the emphasis of urban wastewater management has shifted from a "least-cost" solution to a "cost-effective" solution.

In this work, the urban wastewater management problem is viewed as a multigroup, multipurpose and multiobjective problem. Procedures have been developed for

formulation of control strategies and for evaluating their costs for accomplishing each purpose independently. If the three purposes are viewed as a part of the overall management plan, then it is possible to take advantage of the complementarities that exist among these purposes in order to reduce the cost of urban wastewater management. Procedures for evaluating multipurpose strategies are also presented. The identification of the cost-effective solution then involves identifying tradeoffs between monetary and nonmonetary costs of various strategies and the selection of a best compromise solution.

The equity question deals with apportioning of the project costs among groups/purposes in a fair manner. Economic analysis indicates that users should be charged the marginal cost of adding them to the plan. However, in single-purpose problems involving several groups and in multipurpose problems, if purpose(s) are accomplished simultaneously, charges equalling marginal costs do not raise enough revenues to cover total project cost. For this reason, it is customary to apportion the cost of a multipurpose or a multigroup plan so that each user pays its marginal or separable cost plus a portion of the joint or nonseparable cost. The conventional techniques for accomplishing this do not necessarily result in an equitable arrangement. Further, they are difficult to use in multipurpose wastewater management. The equity problem has

been viewed in the context of N-person cooperative game theory. It is shown that these concepts when applied to cost sharing/cost allocation have desirable equity properties.

CHAPTER 1

INTRODUCTION

Over the last century, the United States has undergone a transformation from an essentially rural to an urban society. In the process, two very definite trends have been established. First, people have concentrated in urban areas. In 1970, two-thirds of the total population was living in Standard Metropolitan Statistical Areas (SMSA's) (Bureau of Census, 1971). The second noticeable trend of concern has been a steady decrease in average urban population density per unit area (Pickard, 1967). Taken together, these two trends result in massive urban sprawl.

During the transition period, streets that were largely unpaved with side swales or ditches that stored appreciable amounts of rain before any runoff occurred were replaced by smooth pavement and curbs. Natural tributary channels and water courses were replaced by smooth, lined underground conduits. Large parcels of undisturbed land were converted into parking lots and shopping centers. These factors combined to create an urban drainage problem of tremendous magnitude. Engineering

response to these drainage needs consisted of intercepting the surface runoff (also called wet-weather flow) as quickly as possible and then conveying this stormwater by means of storm sewers to the nearest water course. These practices caused tremendous increases in the peak stormwater runoff discharged to the receiving water resulting in stream bank overflows, increases in frequency of flooding, and instream erosion due to high velocities.

In addition to stormwater runoff, cities have also been traditionally concerned with disposal of domestic wastes (also called dry-weather sewage flow). Continuous discharge of relatively small concentrated volumes of dry-weather sewage is of concern to public health officials as it contains infectious bacteria. The strategy has been to dispose of this wastewater in the least expensive manner. At the outset, sewers designed for stormwater runoff were utilized to carry dry-weather sewage flow and reliance was placed on the capability of surface waters to provide enough dilution to this water to prevent any health hazard. As the areas increased in size, the dilution capability of surface waters was overtaxed. Objections from citizens and public health officials forced cities to install treatment facilities at the end of the sewer network to provide a minimum level of treatment to the dry-weather sewage flow. These facilities were sized to accommodate two to five times the average

dry-weather flow. This excess capacity was provided primarily to accommodate the normal daily or seasonal variations in the dry-weather flow. If, in the event of a rainfall, the rate of flow was larger than the treatment capacity, the excess flow containing a mixture of untreated sewage and stormwater was bypassed directly to the receiving water. This phenomenon is frequently referred to as a combined sewer overflow.

As public awareness of water pollution grew, the Water Quality Act of 1965 was enacted. Under the Act, the first step was to establish the "beneficial use" to which a particular reach of water was to be put. Then water quality standards that would protect this use were established. The pollution budget was then allocated, ignoring non-point sources (wet-weather flows) as well as future growth patterns, among various pollutors in the area. Cities and industries were then issued waste discharge permits, allowing them to discharge pollution up to a certain specific amount. Since this Act, in general, required a higher level of treatment to dry-weather flow, communities began to investigate coordinated or regional wastewater treatment strategies. The objective is to take advantage of the economies of scale in wastewater treatment and/or less restrictive discharge requirements of some locations. However, there has been little success in implementing such proposals due partially to the

nonexistence of a real-world regional authority with necessary power to shift decisions in this direction.

During the late 1960's, concern was expressed regarding the discharge of combined sewer overflows since, during storms, untreated combined sewage (mixture of dry- and wet-weather flows) is directed to the receiving water (Burm and Vaughan, 1966). Cities were installing separate sewer systems in newly developing areas to carry polluted domestic wastewater and supposedly clean stormwater in separate pipes. Also, programs were initiated to separate the existing combined sewer systems. However, preliminary cost estimates indicated that this separation program would be very expensive (American Public Works Association, 1967). In addition, concurrent stormwater sampling programs indicated that stormwater from urban areas contains significant quantities of pollutants picked up from street surfaces, parking lots and drainage structures (Benzie and Courchaine, 1966). Uncontrolled discharge of these flows may have substantial impact on receiving water quality. It has recently been shown that, approximately 20 percent of the time, receiving water quality in an urban area is controlled by stormwater runoff (Colston, 1974). Thus, sewer separation may not be very cost effective. Recognition of this situation has led many communities to institute controls on stormwater quantity and quality. As an example, regulations of Orange County, Florida state that treatment

is required for stormwater in all drainage systems to reduce pollutants and other objectionable material contained in the stormwater runoff (Orange County, 1972).

A great deal of controversy presently exists on the question of treatment of stormwater and combined sewer overflows versus treatment of dry-weather sewage flow beyond secondary treatment. Studies have shown that peak stormwater runoff after development ranges from three to eight times the predevelopment peak runoff (Carter, 1961; Waananen, 1961). Other recent studies (Field and Lager, 1974) indicate that

(1) Biochemical oxygen demand (BOD_5) concentrations of stormwater approximately equal the strength of domestic sewage after secondary treatment from the same land use. For combined sewer overflow, the BOD_5 loadings average approximately one-half the strength of untreated domestic sewage.

(2) The total suspended solids (TSS) content of the stormwater runoff is generally about three times that of untreated sewage but consists mostly of inorganic materials.

(3) Bacterial coliform content of runoff is about two or four orders of magnitude smaller than untreated sewage. However, it is two to five orders of magnitude higher than is considered safe for water contact recreation.

Research has also revealed that a "first flush" phenomenon is exhibited during the initial periods of a storm event. Many of the pollutants in stormwater runoff are carried off during this period (Bryan, 1972). This is also true for

combined sewer overflow (Lager and Smith, 1974). Due to these factors some people have argued that it may be more cost-effective to treat combined sewer overflow and/or urban stormwater runoff (Bryan, 1972). This question remains unresolved. The management of water quality in urban areas is further complicated by the fact that these areas usually encompass more than one city and the wastewater problems and management goals of each city within the area differ. As a result of all the above factors a general degradation in the water quality in urban areas continues.

Because of the public health aspects of water quality, states and the federal government have played the role of regulating agencies. Subsidies have been provided to the local governments as an incentive to control dry-weather flows. By enacting Public Law 92-500, Congress initiated the most comprehensive program against water pollution the nation has ever experienced. Known as the Federal Water Pollution Control Act Amendments of 1972, the Act has two general goals (1) to achieve where possible, by July 1, 1983, water that is clean enough for swimming and other recreational uses, and clean enough for the protection and propagation of fish, shellfish and wildlife; and (2) by 1985, to have no discharge of pollutants into the nation's waters. Attainment of these goals is expected to require commitments of

national resources far in excess of those ever committed before to water pollution control. In a recent report to Congress, the costs of wastewater treatment plants required between now and 1983 are estimated at $83 billion. Another $235 billion is estimated for controlling stormwater systems (Environmental Protection Agency, 1974). Public Law 92-500 provides for 75 percent construction grants for water pollution control works.

To encourage efficient use of national resources in accomplishing the above goals, Public Law 92-500 requires preparation of Basin Plans (Section 303), Area-wide Waste Treatment Management Plans (Section 208), and Facility Plans (Section 201). The Basin Plan is a management document, which identifies the water quality problems of a particular basin and sets forth an effective remedial program to alleviate those problems. It is neither a broad water and related land resources plan nor a basin-wide facilities plan. The value of the basin plan lies in its utility in making water quality management decisions on a basin-wide scale. The Area-wide Waste Treatment Plan is a document designed for water quality management in those urban areas, within the basin, having a substantial water quality problem. The plan is directed towards meeting the 1983 goal of the Act. It identifies anticipated municipal and industrial treatment works construction over a 20-year period. It also identifies urban runoff and

other non-point sources of pollution and methods to control these sources to the extent feasible. The plan also sets out construction priorities over the next five-year and 20-year periods and designates agencies necessary to construct, operate and maintain the control facilities. It is also a mechanism which will be used for regulatory purposes and for issuing waste discharge permits. The Facilities Plan is a document similar to the area-wide plan except that it is much narrower in scope and detailed consideration of non-point sources is not required.

This work is motivated by the ongoing research at the University of Florida on management of stormwater and combined sewer overflows. The research problem is to develop (1) a decision making model for formulation and evaluation of a set of strategies for controlling dry-weather and wet-weather flows in order to identify a cost-effective solution to urban wastewater management problems in the context of Section 208; and (2) procedures for apportioning the costs of the selected strategy amongst various project purposes and groups in a fair and equitable manner in order to enhance the implementation feasibility of that strategy. In this work, the problem of urban wastewater management is viewed as a multiobjective, multipurpose as well as a multigroup problem as opposed to single purpose, single group problems considered in the past. While the single purpose, single group problems have traditionally been

solved for least-cost, multipurpose/multigroup problems need to evaluate equity as well as efficiency. These objectives are demonstrated by means of examples presented in Sections 3.1 and 3.4. Mathematical formulations of the urban wastewater management problems incorporating efficiency/equity are presented in Section 3.5. New procedures for attaining efficiency as well as equity are presented in Section 7.4 and demonstrated by means of examples in Section 7.5. Single purpose, single group analysis for determining a least-cost solution have been generalized and extended so that alternative strategies can also be formulated and evaluated. The procedures for accomplishing this for domestic wastewater management are discussed in Section 4.1 and for wet-weather quantity control in Section 5.1. For wet-weather quality control, due to its recent importance and lack of sufficient data, it is necessary to develop performance and cost data for various control devices in addition to the procedures for formulation and evaluation of strategies. The performance data are developed in Section 2.3 and the cost data in Section 2.4. Procedures for formulation and evaluation of wet-weather quality control strategies are developed in Section 5.4. In Section 6.1, engineering concepts that may be utilized to increase the treatment capabilities of the dry-weather facilities are presented in order that these facilities may be used for wet-weather quality control. Procedures for evaluation of various multipurpose strategies are discussed in Sections 6.3 and 6.4.

Chapter 2 presents an overview of Section 208 of the 1972 Act Amendments, characteristics of wastewater and procedures for estimating these parameters. A review of various control devices is also included and cost functions for these devices are developed. A brief description of Basin Plans establishing control criteria is presented and data on a hypothetical planning area are outlined.

Chapter 3 presents an overview of the planning objectives discussed in this work. Economic analysis is presented first. Subsequently, multiobjective planning is discussed and the need for efficiency and equity in wastewater management is outlined, and mathematical formulations are developed.

Chapter 4 presents optimization procedures for determining the resource costs associated with various strategies for domestic wastewater management. The use of these procedures is illustrated by solving the domestic wastewater management problem of the hypothetical planning area.

Chapter 5 presents procedures for formulation and evaluation of strategies for wet-weather control. The wet-weather quantity control problem is discussed first. Techniques for determining resource costs associated with various wet-weather quantity control strategies are presented. Subsequently the wet-weather quality problem is

discussed and optimization and evaluation procedures for wet-weather quality control strategies are described. The wet-weather control problem of the hypothetical planning area is solved to illustrate the various concepts.

Chapter 6 discusses procedures for multipurpose planning for water quality control. The concept of joint dry-wet weather treatment is presented and optimization procedures for evaluating the resource costs for various multipurpose strategies are outlined.

Integrated efficiency and equity analysis is presented in Chapter 7. Multiobjective solution techniques as well as existing cost sharing/cost allocation techniques are discussed. A brief review of cooperative N-person game theory is presented. Subsequently, application of these concepts for resolving the equity questions is outlined and its relationship to existing cost sharing/cost allocation procedures is highlighted.

Lastly, Chapter 8 summarizes the various concepts discussed in this work.

CHAPTER 2

URBAN WASTEWATER MANAGEMENT—AN OVERVIEW

2.1 Introduction

Section 208 of the Federal Water Pollution Control Act (FWPCA) Amendments of 1972 emphasizes the development of sophisticated area-wide waste management systems which will result in the successful management of water quality at the substate level in highly complex areas of urban-industrial concentrations. An urban-industrial concentration is that portion of a standard metropolitan statistical area (SMSA), or those portions of SMSAs, having substantial concentrations of population and manufacturing production or other factors which result in substantial water quality control problems. In such areas, water pollution sources can be categorized as point and non-point sources. A point source discharges its effluent into the water body through a pipe or conduit, e.g., municipal wastewater treatment plant or industrial waste effluents. A non-point source is diffuse; it either seeps into the ground through the soil or is carried over the surface of the land by rainwater. Runoff from urban areas,

construction sites, farms, forest lands, and strip mines produces non-point source pollution. The stated objectives of area-wide waste treatment management are to provide (Federal Register, 1973)

(1) cost-effective point source treatment and control;

(2) control of non-point sources; and

(3) coordinated wastewater management.

Pollution emanating from industrial wastes, construction sites, farms, forest lands and strip mines is highly site specific. Thus, control of those sources would depend upon local conditions. Dry-weather sewage flow and urban stormwater are the only two sources for which generalized data can be used for estimating volumes and pollutant loadings. Further, a generalized approach can be used to formulate and evaluate control strategies for these sources. Thus, in this work only these two wastewater sources will be considered.

Designation of area-wide planning boundaries, the key first step in implementing Section 208, is based on the following criteria (Federal Register, 1973):

(1) The area must have substantial water quality control problems, i.e.,.when water quality has been degraded to the extent that desired uses are impaired or precluded.

(2) The area has in operation a coordinated waste treatment management system or the local governments within the area show their intent to develop and implement a plan which will result in coordinated waste treatment management.

Following the designation of the area, it is necessary to inventory existing wastewater sources, estimate present and future wastewater volumes and pollutant loadings, develop cost data on various wastewater control devices and determine the extent of controls that would be required to alleviate or prevent water quality problems. Based on this information various control strategies can be formulated and evaluated to determine a cost-effective solution.

In this chapter various pollutants that are of significance will be described and cost data on various control devices will be presented. Criteria for determining the extent of controls required will be outlined. Data on a hypothetical planning area will then be presented in order that the concepts outlined in later chapters may be demonstrated in the context of this planning area.

2.2 Wastewater Characteristics

Several parameters are used to define the characteristics of domestic wastes and stormwater. The four most widely used parameters are wastewater volume, biochemical oxygen demand (BOD), suspended solids (SS) and coliform organisms. Other parameters of importance include dissolved oxygen (DO), temperature, pH, nitrogen (N) and phosphorous (P).

The volume of wastewater is important for three reasons. First, it determines the size of the conveyance and treatment facilities. Secondly, it is needed to determine the mass loadings of pollutants. Finally, controls on quantity may be required with or without controls on quality. The latter is especially true for wastewater from urban stormwater runoff where control of flooding may be one of the objectives of planning.

The BOD of wastewater is a measure of its strength in terms of quantities of oxygen required to biochemically oxidize the organic matter contained in wastewater. When a wastewater is released, the dissolved oxygen content of the receiving water is utilized to satisfy the biochemical oxygen demand of wastewater thereby depressing or even depleting the DO of the receiving water. The DO of the receiving water is a major parameter which determines its beneficial use. Wastewater BOD is usually expressed in terms of the amount of oxygen utilized during a five-day period (BOD_5). The higher the BOD_5, the more damaging the waste is to the receiving water. To date, BOD_5 is perhaps the most important parameter used for evaluating wastewater control strategies.

Suspended solids refer to the solids which can be mechanically filtered from wastewater. The suspended solids render the wastewater unsightly and unless removed, tend to settle in the receiving water.

Coliform organisms are organisms of intestinal origin and are present in large amounts in municipal sewage. Their presence in a water supply source is an indication of pollution from human sources. Coliform levels are commonly reported as the most probable number of organisms per 100 milliliters of sample (MPN). Sewage requires disinfection before being discharged into the receiving water.

Nitrogen and phosphorous content of the wastewater is important when the receiving water body is a lake where continuous discharge of nitrogen and phosphorous results in its fertilization. This is characterized by explosive "blooms" of algae of such intensity that clear, sparkling lakes are transformed to turbid colored bodies of water. Decomposition of the algae then produces obnoxious odors and floating decomposing mats of organic matter.

The volume of domestic wastes and the quantity of pollutants originating within an area is a function of land use, population density, market value of the dwelling units, use of garbage grinders and characteristics of water supply. Typical values of various parameters of dry-weather sewage flow are listed in Table 2-1. Assuming an annual average domestic wastewater flow of 100 gallons/person-day and an annual average domestic wastewater BOD_5 of 0.17 pounds/person-day, the following equations result:

Table 2-1

Typical Values of Parameters for Domestic Sewage

Parameter	Typical Values
Volume	100 gallons per capita per day
BOD_5[a]	200 mg/l
SS[a]	200 mg/l
P[a]	10 mg/l
N[a]	40 mg/l
Coliforms[a]	5×10^7 MPN/100 ml

[a]Lager and Smith, 1974.

$$DWF = 1.34\ (PD) \tag{2.1}$$

$$DWB = 62.1\ (PD) \tag{2.2}$$

where DWF = annual quantity of dry-weather flow, in/year;

DWB = annual quantity of dry-weather BOD_5, lbs/ac-yr; and

PD = population density, persons/acre.

The volume and pollutant loadings resulting from wet-weather flows are a function of the total area, type of land use, population density, area under each land use,

type of conveyance system such as separate, combined or surface, antecedent storm conditions and amount and duration of rainfall. For wet-weather flows, one is concerned not only with the peak flow and the pollutant loadings but also with their temporal variations. With present-day technology, characteristics of wet-weather flows can only be estimated roughly. The procedure involves the use of simulation models which describe the rainfall-runoff-quality process. Brandstetter (1975) and Brown et al. (1974) assess available models. Two of these models, the U.S. Environmental Protection Agency (EPA) Storm Water Management Model (SWMM) and the Corps of Engineers model, STORM, will be mentioned briefly because of their general acceptance within the engineering community.

The SWMM, developed in 1969-1970 (Environmental Protection Agency, 1971), was formulated as a design model in that it simulates a single storm event. This is characterized by short time steps and total simulation times (i.e., minutes and hours, respectively) and relatively high degree of detail in catchment schematization. The RUNOFF block of the SWMM predicts surface runoff quantity and quality from a given storm event. The TRANSPORT block routes the flow and pollutants through the conveyance network. The output from the TRANSPORT block provides data on the quantity and quality of wet-weather

flows generated from the planning area during each time step. The SWMM also has the capability of simulating areas with combined sewer systems. In addition, various wet-weather control devices can also be simulated using the STORAGE/TREATMENT block.

Within the RUNOFF block of the SWMM, quantity and quality routing occurs in two distinct segments, initially as overland flow and then, if designated, as gutter/pipe routing. The input data required to generate the hydrograph and pollutograph include the area, fraction imperviousness, ground slope, roughness factors, surface depression storage, overland flow width, hyetograph of precipitation and the infiltration coefficients of Horton's exponential function. Pollutant loads are introduced into the stormwater outflow of the RUNOFF block through two mechanisms, the surface pollutant load and catchbasin BOD_5 load. Eight pollutants are modeled by the RUNOFF block: BOD_5, SS, total coliform, COD, nitrogen, phosphorous, settleable solids and grease. For simulating sewer systems and treatment devices, additional appropriate data are included.

In contrast to SWMM, STORM (Hydrologic Engineering Center, 1975) is a planning model capable of continuous simulation in hourly time steps for simulation periods of many years. Hourly runoff volumes are computed as the difference between rainfall and surface storage, multiplied

by the runoff coefficients. There is no pipe or gutter routing in the STORM program. Modeling of runoff quality is based on a percentage washoff of available pollutants as a function of runoff rate. The treatment system is simulated as a black box with its performance specified by the user.

The quality prediction techniques found in most simulation models (e.g., SWMM, STORM) rely upon generation of an initial surface load of pollutants. This load is usually expressed in units of lbs, lbs/acre, lbs/curb-mile, lbs/day-acre, or lbs/day-curb-mile. Normalized loads are, of course, multiplied by a unit of area, dry days, etc., to produce an initial mass of pollutants at the start of the storm. Pollutants are then "washed off" during the storm in an exponential fashion, in which the amount removed per time step is proportional to the amount present, the runoff rate and other factors. Based on thorough review of stormwater sampling studies done during the past 20 years, Heaney et al. (1976) have updated the pollutant loading estimates. The results simplify the original loading factors by eliminating the use of dust and dirt and curb miles. They propose the equations listed in Table 2-2 for predicting annual average loading factors as a function of land use, precipitation and population density. Based on their findings that the fraction of the developed land under residential, commercial,

Table 2-2

Wet-Weather Pollutant Loading Factors
(Heaney *et al.*, 1976)

Separate Areas: $M_s = \alpha(i,j) \cdot P \cdot f_1(PD) \; \frac{\text{lb}}{\text{acre-yr}}$

Combined Areas: $M_c = \beta(i,j) \cdot P \cdot f_1(PD) \; \frac{\text{lb}}{\text{acre-yr}}$

where M = lb of pollutant j generated per acre of land use i per year;
P = annual precipitation, inches;
PD = population density, persons per acre; and
α, β = factors given in table below

Land uses:
i = 1 Residential
i = 2 Commercial
i = 3 Industrial
i = 4 Other (assume PD = 0)

Pollutants:
j = 1 BOD_5, Total
j = 2 Suspended Solids (SS)
j = 3 Volatile Solids, Total (VS)
j = 4 Total PO_4 (as PO_4)
j = 5 Total N

Population Function:
i = 1 $f_1(PD) = 0.142 + 0.533 \cdot PD^{0.145}$
i = 2,3 $f_1(PD) = 1.0$
i = 4 $f_1(PD) = 0.142$

Factors α and β for Equations: Separate factors, α, and combined factors, β, have units lb/acre-yr-in. To convert to kg/ha-yr-cm, multiply by 0.442.

	Land Use, i	Pollutant, j: 1. BOD_5	2. SS	3. VS	4. PO_4	5. N
Separate Areas, α	1. Residential	0.799	16.3	9.48	0.0336	0.540
	2. Commercial	3.20	22.2	14.0	0.0757	0.296
	3. Industrial	1.22	29.1	14.4	0.0705	0.276
	4. Other	0.113	2.70	2.60	0.00994	0.0605
Separate Areas, β	1. Residential	3.20	67.2	38.9	0.139	0.540
	2. Commercial	13.2	91.8	57.9	0.312	1.22
	3. Industrial	5.00	120.0	59.4	0.291	0.140
	4. Other	0.467	11.1	10.8	0.0411	0.250

industrial and open space uses is fairly constant (0.584, 0.086, 0.148, and 0.182, respectively), an average pollutant loading factor over all lands uses can be derived by using these fractions as weighting factors. The resulting equation for predicting BOD over all land uses is as follows:

$$M = K_A(0.467\ P\ \ (0.142 + 0.533PD^{0.145}) + 0.459P) \qquad (2.3)$$

where M = average annual BOD loading over four land uses, lbs-BOD/ac-yr;

P = annual precipitation, in;

PD = population density; and

$$K_A \begin{cases} = 1 \rightarrow \text{storm or unsewered area, or} \\ = 4.12 \rightarrow \text{combined sewer area.} \end{cases}$$

2.3 Wastewater Control Devices

The primary concept upon which the control of domestic wastes has been predicated is the matter of BOD_5 reduction. Therefore, dry-weather flow control devices have been built around satisfying the oxygen demand of degradable organic matter before releasing it to the receiving water. In general, the efficiency of BOD_5 removal of a dry-weather control device is a function of the detention time. The performance of dry-weather control

devices can be characterized by the following simplified equation:

$$c_t/c_o = 1 - e^{-kt} \tag{2.4}$$

where c_t = outlet concentration of BOD_5, mg/l;

c_o = inlet concentration of BOD_5, mg/l;

k = BOD_5 removal rate constant, time^{-1}; and

t = detention time in control device, time.

A dry-weather control device is usually comprised of a combination of treatment processes or levels placed in series. A combination of storage and treatment is considered cost-effective only when the dry-weather flow contains highly variable quantities of industrial wastes. In actuality, dry-weather flow, in itself, undergoes daily fluctuations. The treatment processes are designed on the basis of average daily flow with provision to accommodate daily mass. Thus, the treatment process is underutilized a part of the time and is overloaded at other times. The performance of the facility fluctuates in accordance with the above. Recently it has been argued that a combination of storage and treatment may be more cost-effective (American Society of Civil Engineers, 1975).

The four different levels of treatment most commonly utilized for dry-weather treatment are called preliminary, primary, secondary and tertiary. Initially, wastewater receives preliminary treatment consisting of

bar racks and grit tanks. This is followed by primary treatment consisting of a gravity sedimentation tank. Primary treatment provides 30-35 percent BOD_5 removal and 55-60 percent SS removal.

When BOD_5 and SS removals in excess of those attainable from primary treatment are required, then it is customary to subject the wastewater to secondary or biological level of treatment. As a general rule secondary treatment includes primary and secondary levels. However, in some cases, primary treatment may be omitted. Several treatment processes are available for accomplishing the secondary level. All of these processes rely on the ability of the microorganisms to utilize as their food waste matter present in sewage. The overall BOD_5 and SS removals from secondary treatment are approximately 85-90 percent and 90-95 percent, respectively.

Tertiary treatment is necessary when reduction in BOD_5 and SS in excess of that accomplished by secondary treatment is required or when a substantial reduction in the nitrogen and/or phosphorous content of the wastewater is desired. Tertiary treatment can be accomplished either by biological-physical, biological-chemical or physical-chemical processes. In the biological-physical and biological-checmical processes, a third level of treatment (either physical or chemical) follows secondary treatment. In the case of physical-chemical treatment, the biological or secondary level is omitted.

Assuming that conventional sedimentation, conventional activated sludge and biological-physical processes are representative of primary, secondary, and tertiary levels, performance data for each of these levels are presented in Table 2-3. Sewage treatment processes are normally designed on the basis of annual average flow and BOD_5 expected at the end of a design period (15 to 20 years). Therefore, provisions are made to hydraulically pass peak flows ranging from two to three times the average flow. A generalized relationship between the flow and the efficiency, for a given design flow, is shown in Figure 2-1. During the initial years of plant operation, the flow and the BOD_5 are less than the design figure and the plant will usually operate at higher efficiency than the design. Additional capacity is frequently added before the actual flow approaches the design flow. Thus, these plants are seldom operated at flow in excess of the design.

A wide variety of control alternatives is available for wet-weather quality control and for improving the quality of wet-weather flows (Field and Struzeski, 1972; Lager and Smith, 1974; Becker et al., 1973). Rooftop and parking lot storage, surface and underground tanks and storage in treatment units are the flow attenuation control alternatives. Wet-weather quality control alternatives can be subdivided into two categories: primary devices and secondary devices. Primary devices take advantage

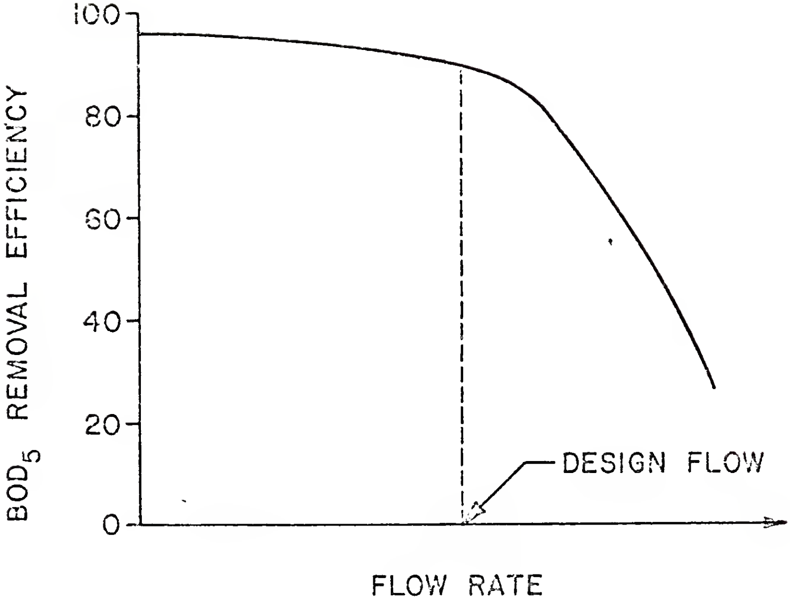


Figure 2-1. Generalized Performance Curve of a Given Size Dry-Weather Plant

Table 2-3

Domestic Wastewater Treatment Performance Data

Control Alternative	Overall Performance[a]		Incremental Performance	
	BOD Removal Efficiency	Detention Time Hours	BOD Removal Efficiency	Detention Time Hours
	%		%	
Primary level	35	2	35	2
Secondary level[b]	86	8	78	6
Tertiary level[b]	95	10	71	2

[a]Overall performance for secondary level includes primary level. Overall performance for tertiary level includes primary and secondary level.

[b]Including secondary clarifier.

of physical processes such as screening (microstrainer), settling and flotation (sedimentation, swirl concentrator and dissolved air flotation). Secondary devices take advantage of biological processes (contact stabilization) and physical-chemical processes (physical-chemical treatment). These control devices are suitable for treating stormwater runoff as well as combined sewer overflows. However, the contact stabilization process is feasible only if the domestic wastewater facility is of an activated sludge type. The quantities of wet-weather flows that can be treated by this process are limited by the amount of excess activated sludge available from the

dry-weather plant. At the present time, there are several installations throughout the country designed to evaluate the effectiveness of various primary and secondary devices. A summary of the design criteria and performance of these devices is presented in Table 2-4. Based on these data, the representative performance of primary devices is assumed to be 40 percent BOD_5 removal efficiency and that of secondary devices to be 85 percent BOD_5 removal efficiency.

2.4 Wastewater Treatment Costs

Costs for dry-weather flow treatment devices have been reported by many investigators. The major references are Smith (1971) and Battelle-Northwest (1974). Cost functions for various levels of domestic waste treatment are presented in Table 2-5. Also included in this table is the cost function for wastewater transmission. All of these functions are of the form $C = 1D^m$, with m less than one, reflecting the economies of scale in wastewater treatment and transmission. Since the control of wet-weather flow has gained attention only recently, well-defined cost data for wet-weather control devices are not available. The costs of various storage and treatment units built around the country for handling wet-weather flows were examined in order to develop generalized cost functions.

Table 2-4

Wet-Weather Treatment Plant Performance Data

Device	Control Alternatives	Design Criteria	Reported BOD_5 Removal Efficiency, η
Primary	Swirl Concentrator[a,b]	60.0 gpm/sq ft	0.25 - 0.50
	Microstrainer[c]	20.0 gpm/sq ft	0.40 - 0.60
	Dissolved Air Flotating w/ Chemical Addition[d]	2.5 gpm/sq ft	0.50 - 0.60
	Sedimentation[e]	0.5 gpm/sq ft	0.25 - 0.40
	Representative Performance		0.40
Secondary	Contact Stabilization[f]	Cont. 0.25 hrs Stab. 3.0 hrs	0.75 - 0.88
	Physical-Chemical[g]	3.0	0.85 - 0.95
	Representative Performance		0.85

[a]Field and Moffa, (1975)

[b]APWA, (1974)

[c]Maher, (1974)

[d]Lager and Smith, (1974)

[e]Performance data based on domestic wastewater treatment

[f]Agnew _et al._, (1975)

[g]Estimate based on performance of these units for domestic wastewater

Table 2-5

Cost Functions for Domestic Wastewater Control[a,b,e]

Treatment Level ψ	Type of Treatment	Annual Cost: \$/yr — Amortized Capital[c] $CA = lD^m$ l_ψ	m_ψ	Operation and Maintenance $OM = pD^q$ p_ψ	q_ψ	Total $TC = uD^v$ u_ψ	v_ψ
1	Primary	61,000	0.70	26,000	0.85	87,000	0.76
2	Primary + Secondary	118,000	0.77	55,000	0.83	173,000	0.79
3	Primary + Secondary + Tertiary	172,000	0.74	88,000	0.85	260,000	0.79

Total Costs for Wastewater Transmission, \$/yr[d] = $11{,}900\ D^{0.51}$

D = Dry-Weather Sewage Flow in million gallons per day (mgd)

[a]Derived from data developed by Battelle-Northwest (1974).

[b]Based on ENR = 2200.

[c]Based on 8 percent interest over 20 years.

[d]Cost per mile. Based on data by Smith (1971).

[e]$D \leq 100$ mgd. No economies of scale beyond 100 mgd.

Cost data for installed wet-weather treatment devices are listed in Table 2-6. Since wet-weather control facilities operate intermittently, annual operation and maintenance costs are greatly affected by the number of hours the facility is utilized. As a general rule, a facility will operate a greater amount of the time if it incorporates storage. An examination of Table 2-6 reveals that annual operation and maintenance costs are 16.7 percent of the total annual costs for the contact stabilization unit. In the case of the swirl concentrator, the percentage is 27.3. Annual operation and maintenance costs for other units fall in between these two values. Based on this analysis, it was decided to assume annual operation and maintenance costs as 20 percent of the total annual costs for all treatment devices. Cost functions developed for various wet-weather quality control devices are presented in Table 2-7. These costs include provisions for sludge handling, engineering, contingencies and land costs.

All treatment units exhibit economies of scale, i.e., $z \leq 1$. Thus, there is an incentive to build larger units. The optimal size treatment unit can be found by comparing the savings in treatment cost of going to a larger unit with the increased piping costs. Unfortunately, sufficient data on the number and flow rate of stormwater discharges in urban areas could not be found. Thus, it is not possible

Table 2-6

Installed Cost for Wet-Weather Treatment Devices

Control Device	Capacity mgd	Annual Cost per mgd: $/yr		
		Amortized Capital[a,b]	Operation and Maintenance	Total
Swirl Concentrator[c]	8.9	5,600	2,100	7,700
Microstrainer[d]	7.4	14,200	3,900	18,100
Dissolved Air Flotation[e]	25.0	71,700	16,700[f]	88,400
Contact Stabilization[g]	20.0	120,000	24,000	144,000

[a]Based on 8 percent interest for 20 years

[b]Construction cost; does not include sludge handling costs.

[c]Field and Moffa (1975)

[d]Maher (1974)

[e]Lager and Smith (1974) for Racine, Wisconsin adjusted to ENR = 2200

[f]Based on 480 hours of operation @ 3.34¢/100 gallons

[g]Agnew et al. (1975). Operation and maintenance costs based on 960 hours of operation.

Table 2-7

Cost Functions for Wet-Weather Control Devices[a,b,i]

Device	Control Alternative	Annual Cost: \$/yr — Amortized Capital $CA = lT^m$ or lS^m		Operation and Maintenance $OM = pT^q$		Total $TC = wT^z$ or wS^z	
		l	m	p	q	w	z
Primary	Swirl Concentrator[c,d,e]	1,970	0.70	580	0.70	2,550	0.70
	Microstrainer[e,f]	7,340	0.76	1,840	0.76	9,180	0.76
	Dissolved Air Flotation[e]	8,160	0.84	2,040	0.84	10,200	0.84
	Sedimentation	32,640	0.70	8,160	0.70	40,800	0.70
	Representative Primary Device - Total Annual Cost - \$4,000 per mgd = \$2610 per acre-inch/hour						
Secondary	Contact Stabilization[g]	19,600	0.85	4,900	0.85	24,500	0.85
	Physical - Chemical[e]	32,640	0.85	8,160	0.85	40,800	0.85
	Representative Secondary Device Total Annual Cost = \$15,000 per mgd = \$9800 per acre-inch/hour						
Storage	High Density (15 per/ac)	51,000	1.00	—	—	51,000	1.00
	Low Density (5 per/ac)	10,200	1.00	—	—	10,200	1.00
	Parking Lot[h]	10,200	1.00	—	—	10,200	1.00
	Rooftop[h]	5,100	1.00	—	—	5,100	1.00
	Representative Annual Storage Cost[j] (\$ per ac-in) = \$122 $e^{0.16(PD)}$						

T = Wet-Weather Treatment Rate in mgd; S = Storage Volume in mg

[a] ENR = 2200. Includes land costs, chlorination, sludge handling, engineering and contingencies.
[b] Sludge handling costs based on data from Battelle Northwest (1974).
[c] Field and Moffa (1975).
[d] Culp et al. (1976).
[e] Lager and Smith (1974).
[f] Maher (1974).
[g] Agnew et al. (1975).
[h] Wiswall and Robbins (1975).
[i] For $T \leq 100$ mgd. No economies of scale beyond 100 mgd.
[j] PD = gross population density, persons/acre.

to determine the optimal mix of treatment plants and pipelines. Therefore, "representative" treatment costs were developed as shown in Table 2-7.

Cost data on detention basins built in the Chicago area for temporary storage of runoff are listed in Table 2-8. Costs of storage tanks built for the purpose of wet-weather quantity and quality control as well as for dry-weather quantity control are also included in this table. Due to the wide variations in these figures, an attempt was made to verify these costs using excavation costs as the basis. Storage costs based on unit excavation costs are listed in Table 2-8. The unit cost of equalization storage basins for sewage treatment plants and the estimated cost of rooftops and parking lot storage are also shown in Table 2-8. Lastly, analysis of recent estimates of storage costs developed by Culp et al. (1976) indicate the following unamortized capital cost C ($\$ \times 10^6$) as a function of storage volume, S (mg).

Type	Equation	Unit Cost @ S = 10 mg $/gal
Earthen	$C = 0.025\ S^{0.73}$	$0.013
Concrete w/o. Cover	$C = 0.350\ S^{0.58}$	$0.133
Concrete w. Cover	$C = 0.400\ S^{0.79}$	$0.250

The data indicate wide variation in the costs of storage. Thus, the relatively simple relationship shown in Table 2-7

Table 2-8

Capital Cost of Storage Facilities[a]

Storage Reservoirs[b]	Capacity mil gal	Capital Cost $/gal	
Hillside Park	11.4	0.01	Earthen Basin
Heritage Park	36.5	0.01	Earthen Basin
Oak Lawn	7.8	0.02	Earthen Basin
Middle Fork North Branch	195.5	0.02	Earthen Basin
Wilke-Kirchoff	32.6	0.03	Earthen Basin
Melvina Dutch	53.8	0.03	Earthen Basin
Oak Hill Park	25.1	0.02	Earthen Basin
Dolphin Park	53.8	0.01	Earthen Basin
Average	52.1	0.019	
Storage Tanks[e]			
Cottage Farm, Boston[c]	1.3	5.21[d]	Covered Conc. Tanks
Spring Creek, New York[c]	10.0	2.33	Covered Conc. Tanks
Chippewa Falls, Wisconsin[c]	2.8	0.29	Asphalt Paved Basin
Humboldt Avenue, Milwaukee[c]	4.0	0.55	Covered Conc. Tanks
Seattle, Washington	32.0	0.25	In-line
Whittier Narrow, Columbus[c]	4.0	1.70	Open Concrete Tanks
Average	9.0	1.72	
Based on Excavation Costs[f]			
$2/cu yd		0.01	Earthen Basin
$5/cu yd		0.025	Earthen Basin in Rock
Equalization Basins for Dry Weather Sewage Treatment Plants[g]	1.0	0.22	Earthen Basin
	3.0	0.10	Earthen Basin
	10.0	0.06	Earthen Basin
	1.0	0.39	Concrete Basin
	3.0	0.29	Concrete Basin
	10.0	0.25	Concrete Basin
Other[h]			
Parking Lots		0.10	
Rooftops		0.05	

[a] Based on ENR = 2200.
[b] Source: Metropolitan Sanitary District of Greater Chicago.
[c] Also used for stormwater treatment.
[d] Includes pumping station, chlorination and outfall facilities.
[e] Source: Lager and Smith, (1974).
[f] Soil Conservation Service, Gainesville, Florida
[g] Flow Equalization - Plus for Wastewater Treatment Plants, American Society of of Civil Engineers, (9/1975)
[h] Source: Wiswall and Robbins, (1975).

was used. Annual storage costs are estimated as a function of population density. The curve was derived using an unamortized capital cost of $0.10 per gallon for PD = 5 persons per acre and $0.50 per gallon for PD = 15 persons per acre.

Since most of the operation and maintenance costs attributable to storage are for solids handling and because these solids are usually handled at the treatment facility, operation and maintenance costs of storage facilities have been assumed at zero.

2.5 Control Criteria

Basin plans proposed under Section 303(e) constitute the overall framework within which 208 plans are developed for specific portions of a basin with complex pollution control problems. Basin plans define (1) water quality standards and goals; (2) critical water quality conditions; and (3) waste load constraints.

In the basin plan, the basin is divided into water quality and effluent limited segments. Water quality limited segments occur when application of best practicable treatment for industrial waste discharges and secondary treatment of domestic discharges would be insufficient to achieve water quality standards. In this case, some significant point sources must be subjected to further control or some non-point sources must be controlled or

some combination of point and non-point source treatment must be implemented. The effluent limited segments are those in which water quality is and will continue to be at least equal to the applicable water quality standards, or if water quality does not meet standards, it will do so after the application of best practicable control technology by industrial sources and secondary treatment by domestic sources. Most of the urban areas requiring 208 planning are expected to contain water quality limited segments.

The basin plans would, in general, specify either the waste load allocation, i.e., amount of pollutants (# of BOD_5 per year, etc.) that can be released from point sources within the area or the minimum degree of treatment that must be provided to these point sources. For non-point sources, such waste load allocation is not expected to be available in basin plans and must be developed during the 208 planning process. This requires water quality modeling studies to determine the extent to which non-point sources must be controlled to meet the water quality standards. Based on these studies control criteria such as the pounds of BOD_5 that can be released on an annual basis can be established.

In addition, stormwater quantity control criteria may be established as a part of the planning process or existing criteria within the region may be assumed to be

followed for future planning. These criteria would be in terms of an allowable rate of release of wet-weather flow.

2.6 Hypothetical Planning Area

The layout of a hypothetical planning area is presented in Figure 2-2. The planning area is assumed to be located in the southeastern part of the U. S. Data on the planning area are listed in Table 2-9. Seven cities are located within the planning area. Existing institutional arrangements are such that cities 1, 2, and 3 belong to one polity; cities 4 and 5 to the second one, and cities 6 and 7 to the third. City 3 has separate sanitary and storm sewer systems while city 6 has a combined sewer system. Both of these cities have existing sewage treatment facilities which neither have any excess capacity nor can be expanded. None of the other cities has a sewer collection or treatment system. The control criteria assumed to be met by each of the cities within the planning area are also listed in Table 2-9.

Methodology for conducting area-wide planning presented in the later chapters will be illustrated by using this hypothetical planning area. Planning objectives involved in area-wide planning are discussed in the next chapter.

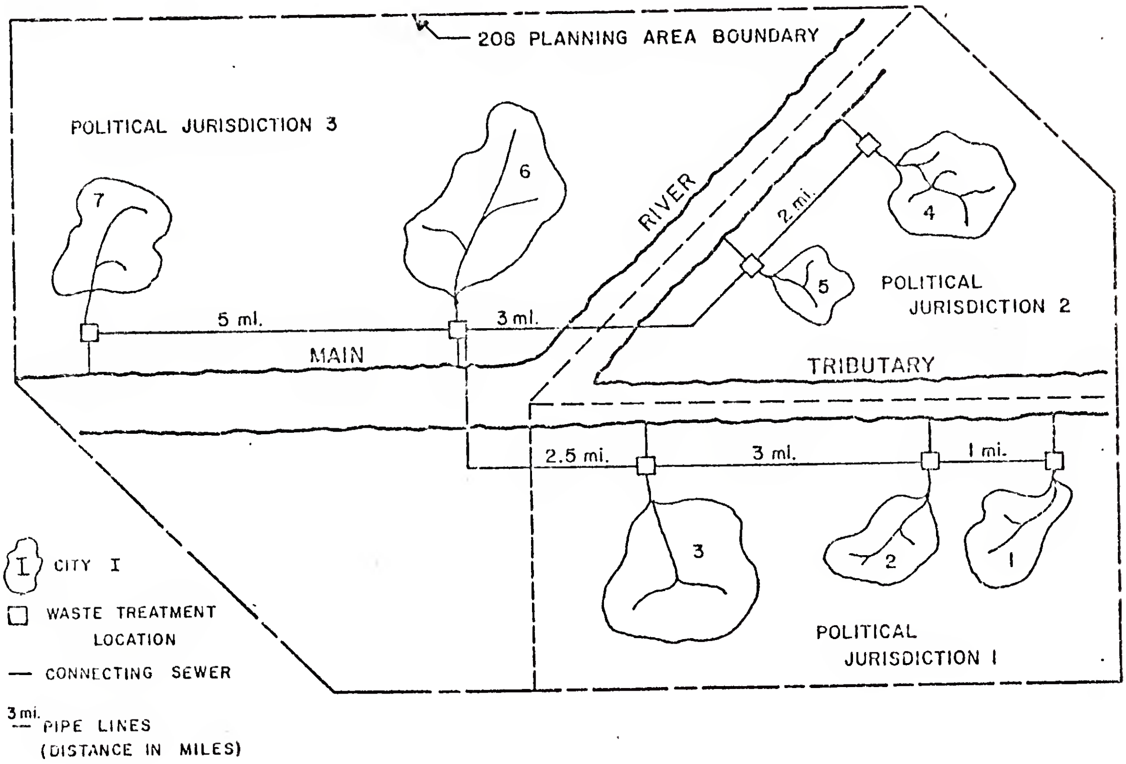


Figure 2-2. Hypothetical Planning Area

Table 2-9

Characteristics of Hypothetical Planning Area

	City						
	1	2	3	4	5	6	7
1. Total Area, Acres	1000	660	4000	2000	400	15000	2000
2. Residential Area and Open Space, Acres							
1975	400	200	1500	800	100	8000	800
1990	550	330	2000	1200	200	10000	1200
3. Industrial and Commercial Area, Acres							
1975	100	30	300	200	15	2000	200
1990	150	60	400	300	25	3000	300
4. Undeveloped Area, Acres							
1975	500	430	2200	1000	285	5000	1000
1990	300	270	1600	500	175	2000	500
5. Population							
1975	2000	1000	15000	5000	500	100000	1000
1990	3000	2000	21000	7000	1000	125000	7000
6. Exist. Sewerage System	-	-	separate	-	-	combined	-
7. Exist. Treat. Plant							
Year Built	-	-	1960	-	-	1960	-
Design Flow, MGD	-	-	1.5	-	-	10.0	-
1975 Flow, MGD	-	-	1.5	-	-	10.0	-
Design BOD_5, #/day	-	-	3000	-	-	20000	-
1975 BOD_5, #/day	-	-	3000	-	-	20000	-
% BOD_5 Removal			30			90	
Type of Facility			(1)			(2)	
8. Projected Waste Loads							
1990 Volume MGD	0.3	0.2	2.1	0.7	0.1	12.5	0.7
1990 BOD_5 #/day	600	400	4200	1400	200	25000	1400
9. Waste Treatment Requirements							
Volume, MGD	0.3	0.2	2.1	0.7	0.1	12.5	0.7
BOD_5 Removal, #/day	540	360	3780	1260	180	22500	1260
10. Control Criteria							
Dry-Weather Flow	(2)	(2)	(2)	(2)	(2)	(2)	(2)
Wet-Weather	(5)	(5)	(5)	-	-	(5)	-
% BOD Removal						30%	

(1) = Primary Treatment

(2) = Secondary Treatment

(5) = Stormwater quantity not to exceed the volume for undeveloped area under 10-year 24-hour rainfall.

CHAPTER 3

FRAMEWORK FOR URBAN WASTEWATER MANAGEMENT PLANNING

3.1 General Statement of the Problem

Urban wastewater planning, under Public Law 92-500, seeks a cost-effective solution for point and non-point sources of water pollution, on an area-wide basis that is politically acceptable and would achieve the 1983 goal of "swimmable and fishable" waters. As stated previously, two main sources of water pollution in urban areas are dry-weather sewage and stormwater runoff. Dry-weather sewage flow is of concern primarily from a quality standpoint. For stormwater runoff, however, both quantity and quality are relevant. Thus, the three facets of urban water management that must be examined in order to achieve the goal of "swimmable and fishable" waters are control of

(1) dry-weather sewage flow quality,

(2) stormwater quantity, and

(3) stormwater quality.

Since the federal government provides construction grants for required facilities, it has set up guidelines for conducting area-wide planning (Environmental Protection Agency,

1975). These guidelines require an integrated approach for evaluating the above three purposes in a cost-effective manner. Cost-effectiveness criteria involve identifying a plan that minimizes "total cost to society," i.e., resource, environmental, and social costs. Resource costs are the values of goods and services representing primary project inputs and include capital costs plus operation, maintenance and replacement costs of water pollution control works required to accomplish the above goal. Environmental and social costs may be viewed primarily as damages associated with the project outputs. In general, there may be numerous strategies for accomplishing all three purposes. Each strategy may have different resource costs as well as environmental and social costs. The cost-effective solution is one which minimizes the total costs.

An urban area will usually embody several separate political entities. Since these groups must bear a portion of resource costs as well as the damages, the selected plan must be acceptable for it to be implementable. Thus, preparation of the area-wide plan involves not only engineering considerations but also economic, financial, social and environmental analysis.

This chapter first reviews basic concepts from economic theory and presents optimality criteria for simple cases of water quality management. Difficulties in applying these criteria are discussed and the need for multiobjective

planning is outlined. Various concepts involved in multi-objective, multipurpose and multigroup planning, in the context of the area-wide plan, are then presented.

3.2 Economic Analysis

Water quality management may be viewed as a production process whose basic purpose is to convert resources from a given form (input) into a more useful form (output). A water quality management project is constructed to produce such desired final outputs as the protection of health, esthetic and other beneficial uses of the water. The immediate output can be assumed to be the amount of pollutant removal. System inputs can be thought of as the physical structures, e.g., treatment plant, storage device, that define system design. These inputs can be directly translated into primary resources such as land, labor and capital.

The relationship between inputs $X = (x_1, \ldots, x_m)$ and outputs $Y = (y_1, \ldots, y_n)$ is expressed by means of a production function

$$g(X,Y) = 0 \qquad (3.1)$$

which identifies those combinations of outputs and inputs by which it is impossible to produce more of one output without either producing less of some other output or requiring more of some input.

Associated with each input and output are the resource costs and the pollutant damage costs.

The economic optimization problem for water quality management can be stated as follows:

$$\begin{aligned} &\text{minimize } Z = F_1(X) + F_2(Y) \\ &\text{subject to } g(X,Y) = 0 \\ &\qquad\qquad\quad X,Y \geq 0 \end{aligned} \tag{3.2}$$

where $F_1(X)$ = sum of resource costs;
$F_2(Y)$ = sum of damage costs; and
Z = objective function.

Thus, the objective is to find an alternative having minimum value of Z with the constraint that only alternatives contained on the production function, $g(X,Y) = 0$, need be considered. Relevant simple cases are examined using a geometrical approach.

1. Single Facility, Single Purpose (one input, one output)

Assume a single treatment plant with its input represented by the BOD removed. This case is typical of dry-weather flow control. The output vector Y is a compilation of the quantities of all goods and services, including labor, required to construct a treatment facility of specified size (flow rate). The output can be represented by a single component, y, the cost in dollars of primary

input resources. The input vector is the amount of various pollutants removed and may be represented by a single component, x, representing BOD_5 removed. The simplified production function becomes,

$$y = g(x) \tag{3.3}$$

where x = pounds of BOD removed per period; and

y = cost of treatment plant of specified capacity in dollars per period.

The resource costs are illustrated in Figure 3-1.

Since the damages to society are incurred as a result of the BOD released, these costs are inversely proportional to the input x (BOD removed or withheld). A generalized curve of these costs vs. the input is shown in Figure 3-1. The objective function is derived by the vertical summation of the two curves. From this curve, the optimal output y^0 and input x^0 is at the point where the resource costs plus the societal damages are at their minimum as shown in Figure 3-1.

2. Dual Facilities, Single Purpose (two inputs, one output)

Assume that we have two facilities such as a storage tank and a treatment plant to accomplish

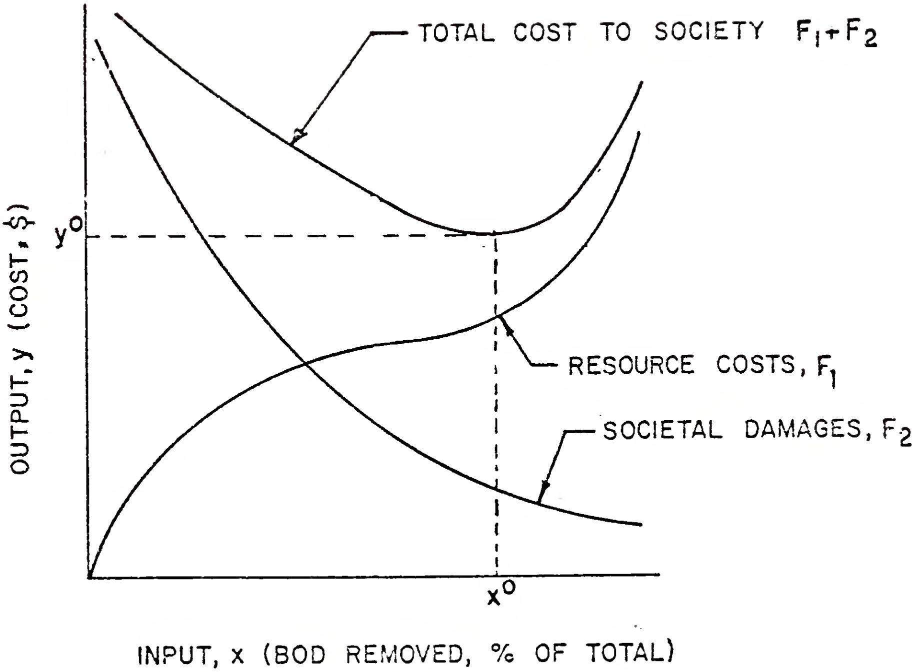


Figure 3-1. Determination of Optimal Level of Input and Output: One Input, One Output

BOD removal. This case is typical for control of wet-weather flows. In this case, the two inputs are the treatment rate and the storage volume. The output is the amount of BOD removed. The production function can be represented by

$$y = g(x_1, x_2) \tag{3.4}$$

where y = pounds of BOD removed;

x_1 = treatment rate; and

x_2 = storage volume.

The above formulation allows isoquants (lines of equal output) to be generated showing all possible combinations of x_1 and x_2 for a specified output level y. Several hypothetical isoquants as a function of treatment rate and storage capacity are shown in Figure 3-2. For a given isoquant, the same output y can be produced by a mix of x_1 and x_2 which yields a given output at the lowest cost. If the costs are linear, then

$$F_1(X) = c_1x_1 + c_2x_2 \tag{3.5}$$

where $F_1(X)$ = total cost;

c_1 = unit cost of resource, x_1; and

c_2 = unit cost of resource, x_2.

Graphically, this shows up on Figure 3-2 as a system

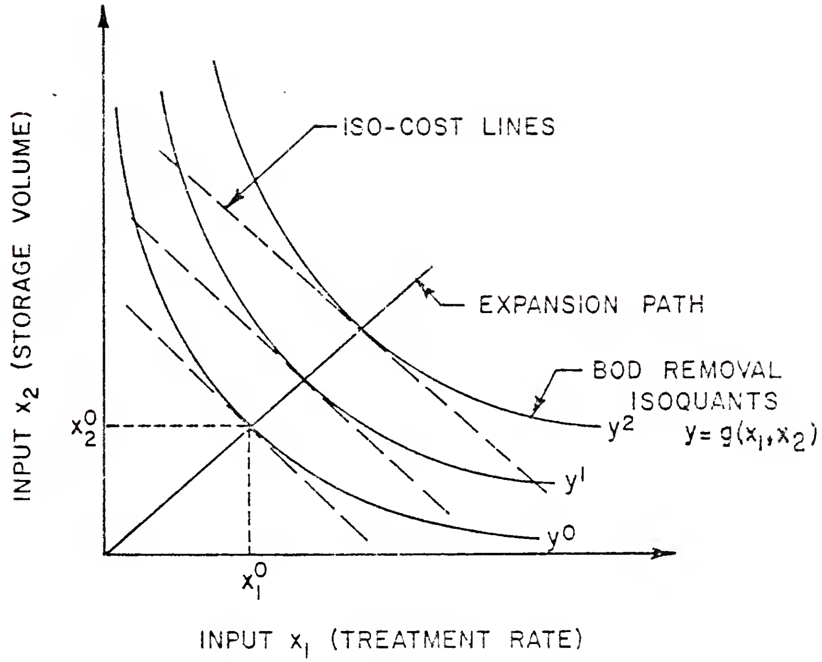


Figure 3-2. Determination of Optimal Combination of Inputs: Two Inputs, One Output

of parallel iso-cost lines. Production of a given level of output, y, with least-cost combination of resources occurs where an isoquant is tangent to the iso-cost line. At this point

$$MRS_{2,1} = \frac{c_1}{c_2} = \frac{MC_1}{MC_2} \qquad (3.6)$$

where $MRS_{2,1}$ = marginal rate of substitution of x_2 for x_1; and

MC_i = marginal cost of input i.

The expansion path, shown in Figure 3-2, traces out the locus of the optimal combination of inputs for various output levels. The expansion path then can be used to derive the curve shown in Figure 3-1, from which the optimal amount of BOD_5 removal can be determined. Lastly, the optimal mix of inputs, x_1° and x_2°, can be determined from Figure 3-2 as illustrated.

3. Single Facility, Dual Purpose (one input, two outputs)

Assume a treatment facility designed to accomplish BOD removals from dry-weather flow as well as wet-weather flow. The input x can be taken as the total resource costs. The two outputs are the amount of dry-weather BOD removed, y_1, and the amount of

wet-weather BOD removed. y_2. The production function becomes

$$x = g(y_1, y_2) \tag{3.7}$$

A family of curves can be obtained showing outputs y_1 and y_2 that can be produced with a given input, x. These product transformation curves, shown in Figure 3-3, identify the locus of all possible combinations of outputs that can be produced with a given fixed input. A concave curve indicates that the two outputs are complementary. The slope of the product transformation curve is called the marginal rate of transformation. A family of parallel lines called iso-revenue lines or iso-benefit lines may also be drawn in Figure 3-3. Each of these lines shows the different combination of outputs that could be obtained for the same amount of gross revenue or benefits. The optimal mix of the outputs is achieved where

$$MRT_{1,2} = \frac{p_1}{p_2} = \frac{MB_1}{MB_2} \tag{3.8}$$

where $MRT_{1,2}$ = marginal rate of transformation of y_i for y_2;

p_j = price for output y_j; and

MB_j = marginal benefits from output, y_j.

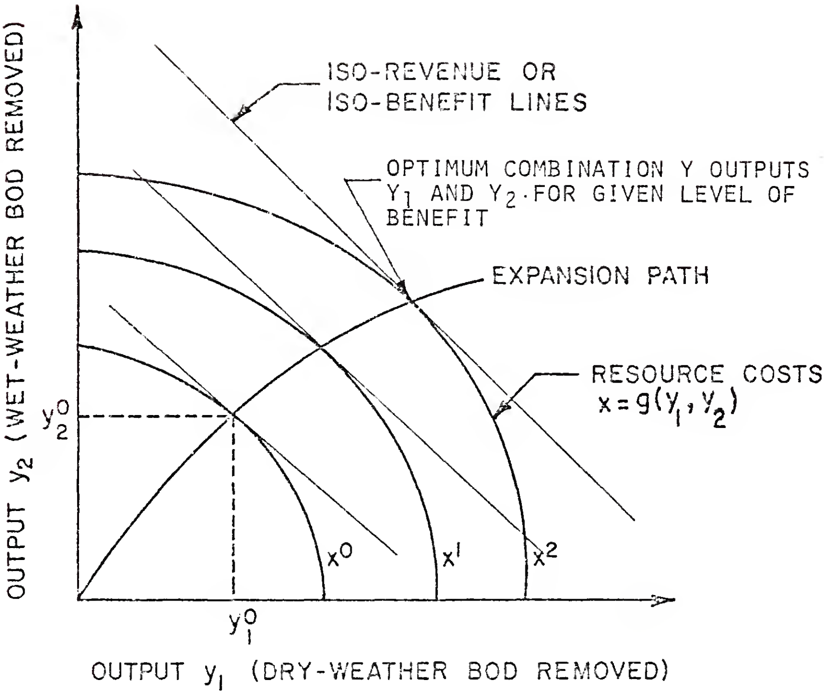


Figure 3-3. Determination of Optimal Combination of Outputs: One Input, Two Outputs

An expansion path showing the optimal combination of y_1 and y_2 for given levels of benefits can be generated only if the output prices or their marginal benefits are known. The expansion path then can be used to construct Figure 3-1, yielding the optimal amount of total BOD_5 removal. The optimal combination of outputs is determined using Figure 3-3.

4. Single Facility vs. Dual Facility

Assume two sources of dry-weather flow are located at some distance from each other. A typical example may be two cities along a river. The problem in this case is to determine whether each city should build its own treatment facility or build one joint facility. Economic analysis required to determine the optimal amount of output (BOD removed) for two single facilities is similar to the single facility, single purpose case shown in Figure 3-1, except that additional environmental and social costs (externalities) may be incurred by one or both cities as a result of the facility of the other community. The optimal joint output, y^1, for two facilities is equal to

$$y_1^1 + y_2^1,$$

where y_i^1 = optimal output (BOD removed) by plant i.

The total resource costs are

$$x^1 = x_1^1 + x_2^1$$

where x_i^1 = resource cost of plant i.

The economic analysis for the joint facility is similar to the single facility-dual purpose case illustrated in Figure 3-3. The optimal joint output is y" and the total resource cost is x". Then the joint facility is cost effective if and only if

$$x'' \leq x_1^1 + x_2^1 = x^1 \quad (3.9)$$

5. Allocation of Capacities Between Purposes

Assume a treatment facility used primarily for dry-weather BOD control and a storage facility used primarily for wet-weather BOD control. The objective in this case is to determine if the total output can be increased by a reallocation of treatment and storage. The storage-treatment isoquants for dry-weather BOD and wet-weather BOD are shown in Figures 3-4 and 3-5. Let T_1° be the available capacity of the treatment facility and S_2° be the available storage capacity.

Conditions that must be fulfilled to guarantee efficient allocation of resources between two consumers

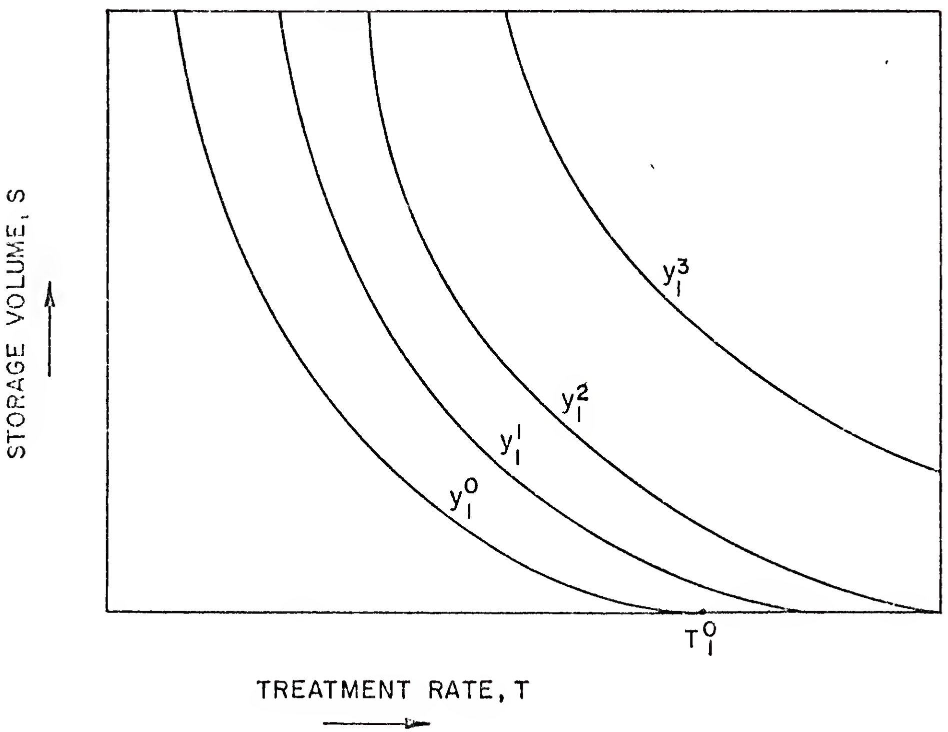


Figure 3-4. Generalized Isoquants for Dry-Weather Control

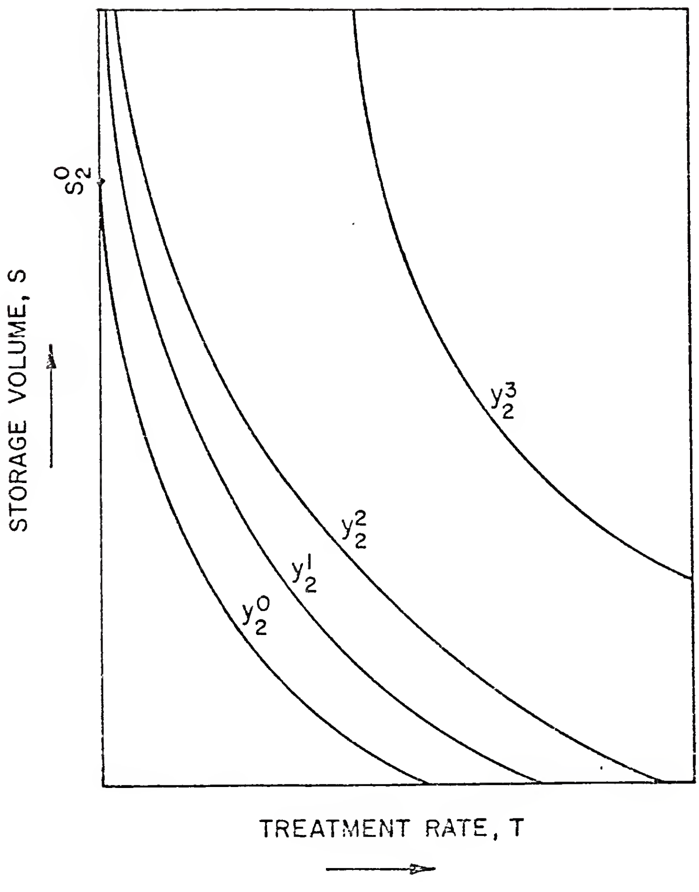


Figure 3-5. Generalized Isoquants for Wet-Weather Control

are usually demonstrated in economic theory by an Edgeworth Box diagram (Leftwich, 1965; Cohen and Cyert, 1965; Bell and Todaro, 1969). Using this procedure, the Edgeworth Box diagram for this problem is presented in Figure 3-6. The dry-weather BOD removal, y_1, increases from bottom left to top right and the wet-weather BOD removal, y_2, increases from top right to bottom left. As can be seen from Figure 3-6, movement along isoquant y_1^0 reallocates storage and treatment such that dry-weather BOD removal remains y_1^0, while wet-weather BOD removal increases. At point B, wet-weather BOD removal is increased from y_2^0 to y_2^3, while dry-weather BOD removal remains constant at y_1^0. At this point, T_1^1 of the treatment capacity and S_1^1 of the storage volume have been allocated to dry-weather BOD removal and the remaining treatment and storage to wet-weather BOD removal. Similarly, if a move is made along isoquant y_2^0, wet-weather BOD removal remains constant while dry-weather BOD removal increases. At point A, dry-weather BOD removal has increased from y_1^0 to y_1^3, while wet-weather BOD removal remains the same. This point corresponds to reallocation of T_1^2 of treatment and S_1^2 of storage volume to dry-weather control and the remainder to wet-weather control. Curve AB is called

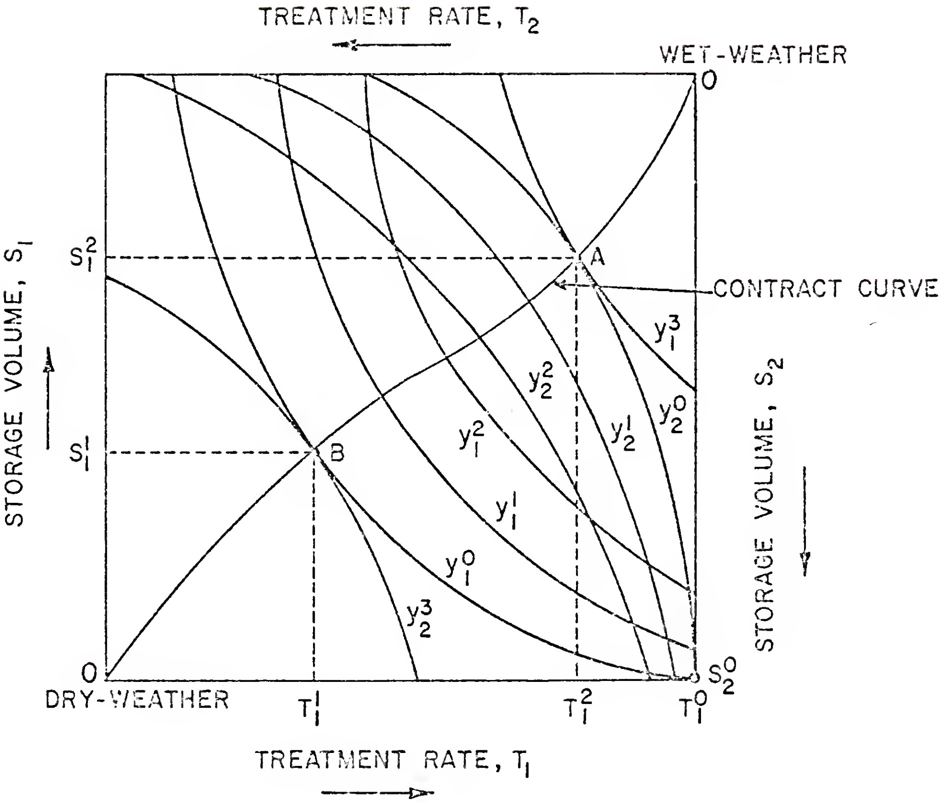


Figure 3-6. Edgeworth Box: Two Inputs, Two Outputs, Two Purposes

the contract curve. It will be designated as the noninferior set because dry-weather BOD removal can only be increased by decreasing the wet-weather BOD removal or vice versa. This is shown in Figure 3-7, which is derived from the contract curve AB. As in case three above, the optimal mix of the two outputs can be determined only if the product price or marginal benefits are known. Knowing the optimal mix of the two outputs, optimal allocation of treatment and storage to the two purposes can be determined from the Edgeworth Box. The above analysis would also apply if the two purposes were to be replaced by two groups such as city 1 and city 2. The results yield the optimal allocation of treatment and storage between these cities.

Application of these economic concepts to water quality management is usually not practical for several reasons. First, it requires the evaluation of environmental and social costs or the "societal damage cost function" which relates project outputs and damages in order to determine the optimal output. These costs are not identifiable in monetary terms. Rather, they are usually described using qualitative and quantitative terms as the adverse environmental and social impacts. Thus, the economic optimization problem of equation 3.1 needs

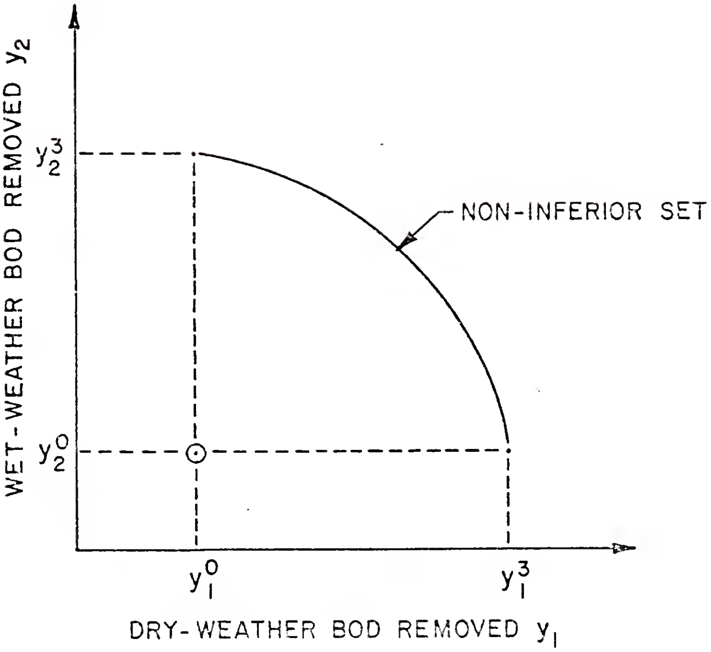


Figure 3-7. Noninferior Set for Two Outputs

to be extended to the following:

$$\text{minimize } Z = (\text{resource cost; environmental and social costs})$$

$$= F_1(x);\ F_2(y) \qquad (3.10)$$

$$\text{subject to } g(x,\ y) = 0$$

$$x,\ y \geq 0$$

The economic analysis above also requires specification of the product prices or marginal benefits in order to determine the optimal mix of outputs. If it is assumed that the marginal benefits from removal of one pound of dry-weather BOD are the same as that obtained from removal of one pound of wet-weather BOD, the marginal benefit curve would be a 45° line and the optimal mix of outputs can be determined. If the marginal benefit curve cannot be specified, then the solution would be the entire set such as shown in Figure 3-7. Further, even where the optimal mix of outputs can be obtained by using the marginal benefit curve, the economic theory derives this solution based on efficiency alone and does not concern itself with whether the allocation is equitable. Thus, the economic optimization problem in this case needs to be extended as follows:

$$\text{maximize } Z = (\text{Dry-weather BOD; Wet-weather BOD})$$
$$= (y_1, y_2) \qquad (3.11)$$
$$\text{subject to } g(S_1, T_1, y_1) = 0$$
$$g(S_2, T_2, y_2) = 0$$
$$S_1, T_1, S_2, T_2, y_1, y_2 \geq 0$$

The problem may be viewed as an efficiency and equity problem.

3.3 Multiobjective Planning

Problems involving objective functions of the form given by equations (3.10) and (3.11) are usually referred to as multiobjective problems as they involve optimization of a vector of objectives. During the last few years, multiobjective planning has been promulgated in the general area of water resources development. Traditionally, projects for water resources development have been evaluated on the basis of a single criterion, national economic efficiency. The procedure involves an evaluation of the total benefits and costs of the project to examine its economic impact. Strong objections have been levied against benefit-cost analysis (Maass, 1970; Prest and Turvey, 1965; Whipple, 1971; Cohon, 1973). The criticism has led to the promulgation of multiobjective planning (Water Resources Council, 1970) using the following four objectives:

(1) national economic development (i.e., national economic efficiency),

(2) environmental quality,

(3) social well being, and

(4) regional development.

Howe (1971) argues that social well-being and regional development can be classified as distributional problems, i.e., that these objectives address the question of how the benefits and costs are distributed. The same is also partially true for environmental quality. These problems are generally referred to as equity questions. Thus, the above four objectives can be narrowed down to two objectives, i.e., efficiency and equity. The objectives to be accomplished in urban wastewater management are quite similar to those for water resources planning. Thus, the multiobjective planning in urban wastewater management may be viewed as an extension of the policy to water quality management.

The general theory of multiobjective planning is outlined by Major (1969) and Howe (1971). While benefit-cost analysis maximizes economic efficiency, multiobjective analysis maximizes a vector quantity, the elements of which are the net benefits associated with each objective. If, for example, the objectives are to minimize the cost of treatment (dollars) and pollution load (estimated as # of BOD_5 discharged) subject to appropriate constraints,

a transformation curve can be generated by successive solution of this vector minimization problem. The transformation curve defines the set of noninferior solutions. This set is comprised of those solutions wherein it is impossible to increase the value of one objective without decreasing the value of the other objective. The transformation curve and the noninferior set are illustrated in Figure 3-8. If one knows the indifference curve (rate of tradeoff between or among objectives), an appropriate mix of these objectives, known as the best compromise solution, can be found as illustrated in Figure 3-8. This solution then yields a best possible distribution of treatment cost and quantity of BOD discharged. Unfortunately it is no trivial matter to determine both curves in actual practice.

A general formulation of the multiobjective problem is as follows:

$$\text{minimize } Z(X) = Z_1(X),\ldots\ldots,Z_p(X)$$

$$\text{subject to } g_i(X) \leq 0 \qquad \forall i \tag{3.12}$$

$$X \geq 0$$

where X = a vector of decision variables;

$Z_i(X)$ = ith objective; and

$g_i(X)$ = constraints.

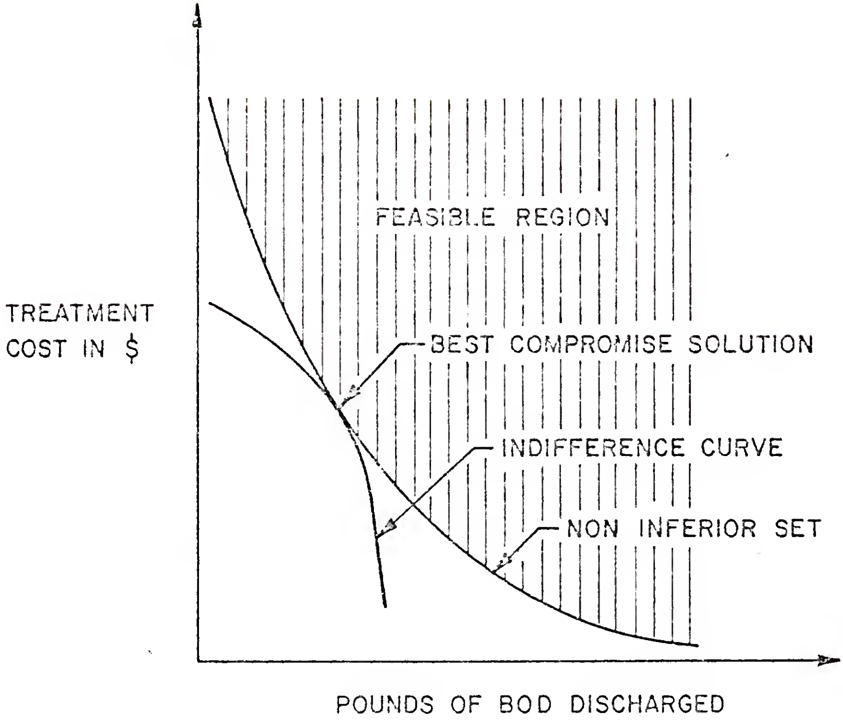


Figure 3-8. Determination of Best Compromise Solution

Mathematical programming techniques offer a promising way of analyzing the above multiobjectives (Cohon, 1973; Cohon and Marks, 1973). By using either weighting techniques or constraint techniques, the transformation curve is generated. In the former case, this is accomplished by varying the weights on each of the objectives. It is easy to do this computationally using parametric programming. The alternative approach optimizes one of the objectives subject to the usual constraint set and an additional constraint which indicates a prespecified level of attainment of the second objective. Thus, from a computational point of view, the noninferior set of solutions can be determined. However, to solve the problem, the rate of tradeoff between objectives needs to be articulated. In actual practice, the set of noninferior solutions is submitted to the decision maker(s) who then selects the best compromise solution. Since the indifference curve is usually not available, selection of the best compromise solution implies value judgements. Such a situation does not ensure that the solution attains the equity objective. Some multiobjective solution techniques for deriving the best compromise solution are discussed in Chapter 7. The next two sections discuss the efficiency and equity questions that arise in the context of urban wastewater management.

3.4 Efficiency and Equity

There has been sustained interest over the past decade in determining optimal regional environmental quality management strategies. Numerous investigators have demonstrated that coordinated wastewater treatment strategies are more efficient, in an economic sense, than decentralized treatment plants, e.g., Heaney et al. (1971). Similar results have been obtained for air pollution, e.g., Teller (1968) and Seinfield and Kyan (1971) and will also be demonstrated to occur in urban waste management. However, there has been little success in implementing such proposals due partially to the nonexistence of a real-world regional authority with necessary power to shift decisions in this direction. Lacking such a regional authority, Hass (1970) investigated the possibility of setting up a system wherein price guides could be used to direct the activities of the individual waste dischargers toward the regional optimum. He structured the problem using the decomposition principle (Dantzig and Wolfe, 1960). Briefly, the decomposition principle partitions the total regional problem into a series of subproblems—one for each waste discharger, and a regional master problem. Each waste discharger submits a provisional control plan to the regional authority who runs the master problem to see if a regional optimum has been achieved. If not, he transmits a revised set of criterion elements, cost coefficients in this case, to the

individual waste dischargers. They resolve their problem and may decide to submit an additional solution for consideration. Each such solution represents an extreme point from their feasible region. Since there are only a finite number of such extreme points, the algorithm eventually converges. The resultant optimal regional solution is actually a weighted average of the extreme points of the solutions submitted by the individual waste dischargers. However, it may occur that the optimal solution for an individual is a convex combination of two adjacent extreme points so that the notion of decentralization by price guides alone breaks down (Baumol and Fabian, 1964). The reason is that the individual is now indifferent among solutions along this edge connecting the adjacent extreme points while the regional authority knows precisely where along the edge the individual should act in the interest of regional efficiency. Thus, in general, more than price guides are necessary to achieve the regional optimum. Charnes, Clower and Kortanek (1967) suggest the inclusion of preemptive goals as a device for providing the requisite amount of information to attain a stable condition.

Dorfman and Jacoby (1970) have argued that the optimization models might be used to screen the number of alternatives down to a reasonable number (say five to ten) and then submit these Pareto-admissible solutions to further scrutiny by employing a simple political

simulation model. The essence of this approach is to assign weights based on the relative importance of each decision-making group. One can then generate various weighting schemes and examine how sensitive the solution is to the assumed weights. Burke and Heaney (1975) have devised a more formalized political simulation model based on work by Bulkley and McLaughlin (1966) and applied it to the Dorfman-Jacoby example. Their effort describes the relative power of several interest groups and simulates the bargaining process and coalition formation among these groups.

An essential component of any workable program is the notion that the resultant solution not only is efficient but also is fair to every one of the participants. Thus, if the performance standards or control criteria are specified, the efficient solution is the least-cost solution subject to meeting this control criterion. Given this solution, the question of equity must then be resolved. Some of the concepts involved in efficiency and equity are outlined below.

There are numerous ways to control pollution using on-site control and/or off-site control. Thus, a general framework is needed for addressing the problem. The selected approach is based on the planning theory of zoning (Herzog, 1969). Using the planning theory of zoning, each source of pollution calculates the cost of handling the

problem on-site as a function of the allowable rate of release from his area. The cost function would look like the curve shown in Figure 3-9. Costs decrease as the allowable release increases. In fact, the cost falls to zero as it is allowed to release more and more pollution. Assume that such a curve exists for each area. Note that the abscissa could be air pollutants, water pollutants, noise or any other "nuisance" which is normally covered by zoning regulations.

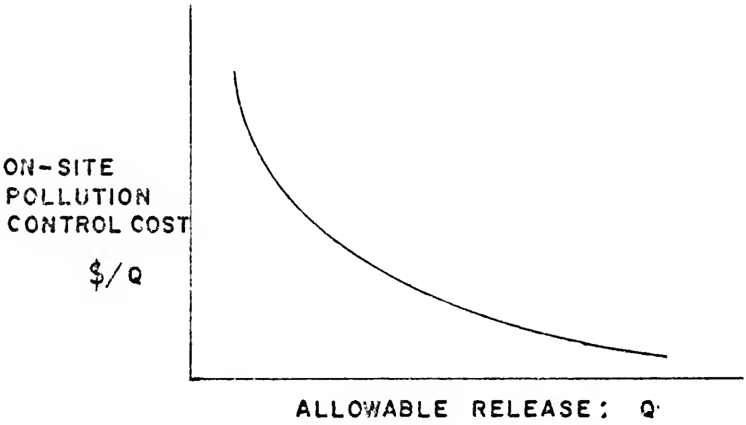


Figure 3-9. Generalized Cost Function for On-site Stormwater Control

In this case, the policy question is to determine how much pollution can be released from each of these areas. Using the planning theory of zoning, the answer depends on a determination of "assimilative capacity." Assume that the required level of control can be specified.

Example

The following example is presented in order to illustrate many of the concepts to be discussed. Assume the region under consideration has been partitioned into three study areas. Each study area has two options: (1) on-site control, and/or (2) off-site control at a central control facility. The following notation is used. Let

x_{ij} = decision variable: number of units of control j selected for area i;

$\overline{x}_{ij}$ = upper bound on x_{ij};

c_{ij} = unit cost for control j in area i;

D_i = quantity of commodity originating in area i;

$\overline{Q}_i$ = maximum allowable release of pollutant from area i;

Q_i = quantity of pollutant released from area i;

Z_i = total control cost to area i;

π_i = reduction in control cost to area i if $\overline{Q}_i$ is increased by one unit;

t_i = unit cost of transporting pollutant from area i to the central control location;

c = unit cost of central control; and

$\overline{W}$ = maximum control at central facility.

The example problem is shown in Figure 3-10 using a network representation. This problem is deliberately oversimplified to permit us to understand the concepts without getting enmeshed in computational difficulties. The results can be extended easily to more realistic cases where multiple central control facilities exist.

The overall objective function seeks to minimize the total cost of on-site and off-site control. The problem facing each study area is to

$$\begin{aligned}
\text{minimize} \quad & Z_i = \sum_j c_{ij}\, x_{ij} + (c+t_i)Q_i \\
\text{subject to} \quad & \sum_j x_{ij} + Q_i = D_i \\
& Q_i \leq \overline{Q}_i \\
& x_{ij} \leq \overline{x}_{ij} \quad \text{for all } j, \\
& x_{ij} \geq 0 \quad \text{for all } j, \text{ and} \\
& Q_i \geq 0
\end{aligned} \tag{3.13}$$

For area 1, the problem is to select x_{11}, x_{12}, and x_{13} so as to minimize $Z_1 = 10x_{11} + 5x_{12} + 0x_{13} + 4Q_1$

$$\begin{aligned}
\text{subject to} \quad & x_{11} + x_{12} + x_{13} + Q_1 = 500 \\
& Q_1 \leq \overline{Q} \\
& x_{11} \leq 100 \\
& x_{12} \leq 200 \\
& x_{13} \leq 200 \\
& x_{11}, x_{12}, x_{13}, Q_1 \geq 0
\end{aligned} \tag{3.14}$$

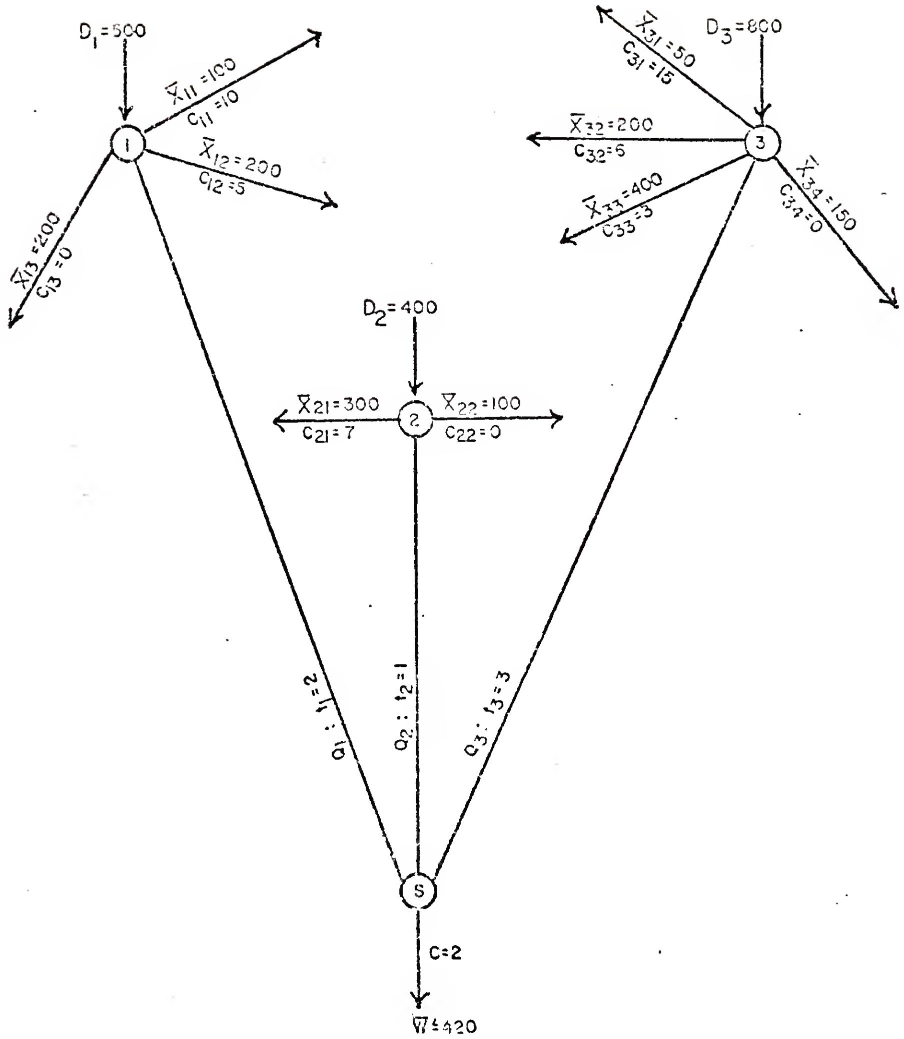


Figure 3-10. Network Representation of Example Problem

This is a very simple problem to solve if $\overline{Q}_1$ is known. As it turns out this is a question of critical importance in some cases. In the context of environmental quality management $\overline{Q}_1$ represents a judgement on the part of the administrator as to the "assimilative capacity" or availability of off-site controls to area 1. Traditionally, the natural system has provided these off-site controls free of charge. For example, the off-site area in this example could be a swamp, river, or the atmosphere.

Willingness to Pay for Off-Site Control

In order to find $\overline{Q}_i$, we need to know how much each area is willing to pay for off-site control based on its alternative on-site control costs. This can be done by solving the above linear program for various values of $\overline{Q}_i$, assuming on-site control is required. Thus, for the moment, $\overline{Q}_i$ is analogous to an effluent standard imposed on area i. The problem can be solved by deleting the last term, $(t_i + c)Q_i$, from the objective function and finding the optimal solution for assumed levels of $\overline{Q}_i$. Computationally this can be done quite easily using parametric programming as explained below. Initially, set $\overline{Q}_i = 0$ and solve the linear programming problem. Then, as a postoptimal procedure, one can vary $\overline{Q}_i$ continuously from 0 up to any prescribed upper bound using as the right hand side $\overline{Q}_i = 0 + \theta r$ where θ equals a parameter which

will increase continuously from 0 to 1 and r is a scalar, say 1000, in this case. This solution to this problem tells the total cost to area i for any value of $\overline{Q}_i$ which is of interest. This is a very attractive feature of linear programming. Another aspect of linear programming which is of interest is duality theory. The solution to the dual problem is obtained when the above problem (primal problem) is solved. Among other things, it tells, for a given $\overline{Q}_i$, the reduction in cost to area i if $\overline{Q}_i$ is increased by one unit. This unit cost is called the "shadow price" with respect to $\overline{Q}_i$ and will be denoted as π_i.

Solve the example problem for area 1 assuming $\overline{Q}_1 = 0$. The answer is simply that on-site control is used. As the constraint on $\overline{Q}_1$ is relaxed, area 1 will substitute off-site control for its most expensive on-site control, x_{11}, in this case. Thus, it is saving \$6 per unit change in $\overline{Q}_1$ in this range. The analysis continues in this manner until all solution possibilities have been identified. The results are shown in Figure 3-11.

Assume that a similar analysis was done for areas 2 and 3. Then the willingness to pay for each of the three areas would be known. Next assume each area was offered as much discharge as it wanted at its cost of $(t_i + c)$ dollars per unit. In this case, the aggregate demand would be $Q_1 = 300$, $Q_2 = 300$, $Q_3 = 250$, or a total demand of 850 units. But suppose only 420 units are

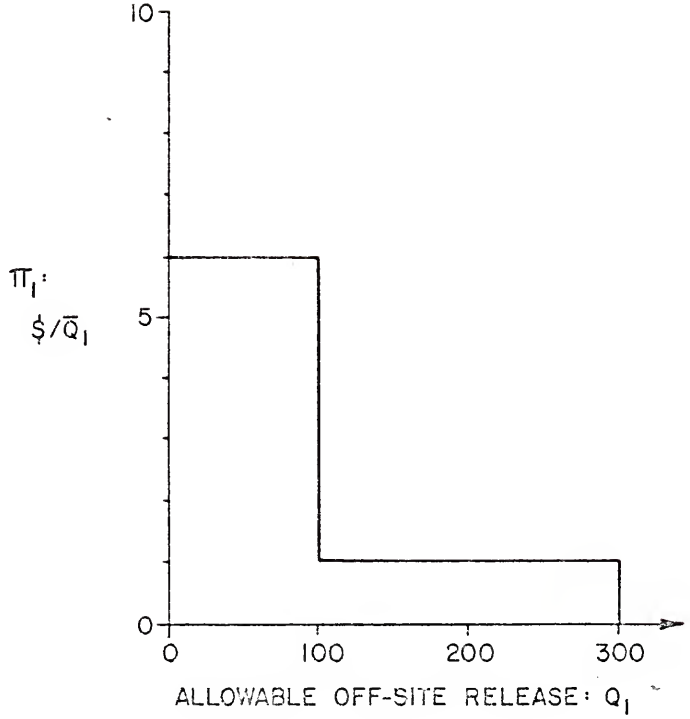


Figure 3-11. Shadow Price for Area 1 for Assumed Value of $\overline{Q}_1$

available. How should the available capacity be allocated? It can be assumed that the central agency or planning authority will attempt to maximize the aggregate savings to areas 1, 2, and 3 from using the regional facility. Alternatively preference will be given to those areas whose on-site control costs are the highest.

Solution of the Regional Problem

The answer to the above question can be obtained by solving one larger optimization model which is formulated on the following page. The constraints for this problem can be divided into two categories: (1) three sets of study area constraints, and (2) a coupling constraint. The coupling constraint is the only linkage among the three study areas. If the values of Q_1, Q_2, and Q_3 are prespecified, then the larger problem can be completely decomposed into three independent subproblems. There are many real-world situations in which an a priori apportionment is used, e.g., allocation to each area based on its size, population, etc. For a pollutant, $\overline{W}$ might be the assimilative capacity of the receiving water, which has been apportioned. In the case of a pollutant, the appropriation is equivalent to prescribing effluent standards.

Assume an apportionment is established such that $Q_1 = Q_2 = Q_3 = 420/3$. Given this apportionment, find Z_1, Z_2, and Z_3, the least-cost solutions for the three areas.

Linear Programming Formulation of the Regional Problem

Minimize Z =	$\sum_{j=1}^{3} c_{1j}x_{1j} + (c+t_1)Q_1$	$+ \sum_{j=1}^{2} c_{2j}x_{2j} + (c+t_2)Q_2$	$+ \sum_{j=1}^{4} c_{3j}x_{3j} + (c+t_3)Q_3$	
Subject to				
Area 1's Problem	$\sum_{j=1}^{3} x_{1j} + Q_1$			$= D_1$
	x_{1j}			$\le \bar{x}_{1j}$ for $j=1,2,3$
	x_{1j}	0	0	≥ 0 for $j=1,2,3$
	Q_1			≥ 0
Area 2's Problem		$\sum_{j=1}^{2} x_{2j} + Q_2$		$= D_2$
	0	x_{2j}	0	$\le \bar{x}_{2j}$ for $j=1,2$
		x_{2j}		≥ 0 for $j=1,2$
		Q_2		≥ 0
Area 3's Problem			$\sum_{j=1}^{4} x_{3j} + Q_3$	$= D_3$
	0	0	x_{3j}	$\le \bar{x}_{3j}$ for $j=1,2,3,4$
			x_{3j}	≥ 0 for $j=1,2,3,4$.
			Q_3	
Coupling Constraints	Q_1	$+ Q_2$	$+ Q_3$	$\le \bar{W}$

(3.15)

The results are

$$Z_1 = \$1360 : \pi_1 = \$1 : \quad Q_1 = 140$$
$$Z_2 = 1540 : \pi_2 = 4 : \quad Q_2 = 140$$
$$Z_3 = \underline{2560} : \pi_3 = 1 : \quad Q_3 = \underline{140}$$
$$\sum_{i=1}^{3} Z_i = \$5460 \qquad \sum_{i=1}^{3} Q_i = 420$$

Apportioning the capacity among the three areas results in a combined cost of \$5,460. Next examine whether, from a least-cost point of view, it would be possible to select Q_1, Q_2, and Q_3 such that costs are reduced. This problem can be solved by running the entire linear program with Q_1, Q_2, and Q_3 as decision variables. This approach is equivalent to receiving system standards wherein the coordinator allocates the assimilative capacity in an optimal manner. This is precisely the problem to be addressed here. The optimal solution is

$$Z_1 = \$1400 \qquad Q_1 = 100$$
$$Z_2 = 1020 \qquad Q_2 = 270$$
$$Z_3 = \underline{2650} \qquad Q_3 = \underline{50}$$
$$\sum_{i=1}^{3} Z_i = \$5070 : \pi = \$4 \qquad \sum_{i=1}^{3} Q_i = 420$$

The solution to the overall optimization problem reduced total costs from \$5,460 to \$5,070 by making more effective

use of the available capacity. As can be seen by examining the solutions, the model took capacity from 1 and 3 and allotted it to 2 since its net savings (π_2) were higher. While this latter solution is the least costly from an overall point of view, if the costs are assigned as shown above, areas 1 and 3 are made worse off while area 2 is made significantly better off. Thus, areas 1 and 3 might reasonably object to such a solution. This is precisely the problem that has thwarted implementation of optimal programs, i.e., while they are the least costly, they do not seem to be "fair" to everyone. Therefore one needs a procedure which is not only efficient but also equitable.

Solution Using Market Price Concept

Perhaps one could use demand theory from economics to determine a better solution. The aggregate demand curve for the three areas is shown in Figure 3-12. Knowing the demand curve and the supply curve, then it is possible to determine the "market price" for the central facility. Referring to Figure 3-12, it is \$4/Q. Thus, according to economic theory, to achieve efficient resource allocation, a market price of \$4 per unit of storage should be used. Let π denote the market price (which is the same as the shadow price of the central facility from the overall optimization model), then the assessment to each study area per unit is $(\pi + c + t_i)$.

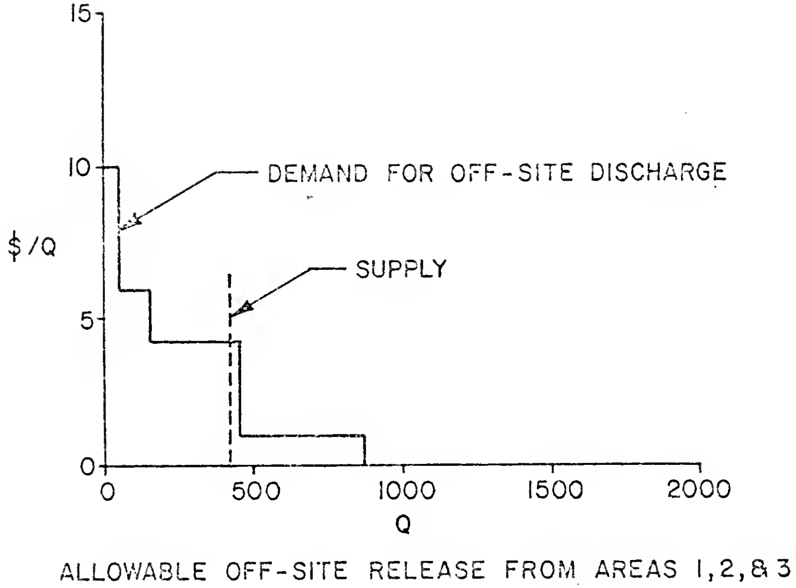


Figure 3-12. Demand for Off-Site Discharge

Economic theory does not address the distributional questions associated with using a market price concept. Thus, until recently, a very fundamental question was left unanswered as indicated below. First, if one actually charges the "market price" then a profit of 420 π is realized. What do you do with the money? Traditionally, public services have been priced based on the cost of the service. Thus, one could argue that such a price cannot be charged. Economic theory uses the notion of consumer surplus which is defined as the difference between an individual's willingness to pay and the actual assessment levied against him.

Value of the Off-Site Disposal

Let us determine the aggregate "consumer surplus" in this case if each area pays for off-site discharge it is allotted. As can be seen from this calculation,

Area	Consumer Surplus if $(c + t_i)$ is Charged
1	6(100) = $ 600
2	4(270) = 1080
3	10(50) = 500
	$2180

the aggregate "consumer surplus" is $2180. In this example, the "consumer surplus" is the savings which accrue to each of the three areas because of the availability of

420 units of off-site disposal. This leads us to an important principle. Using this approach, it is possible to impute a value to the off-site facility based on the services it is performing which otherwise would have been required to be installed on-site. If the central facility is part of the natural system, say a swamp, flood plain, river, lake, or the atmosphere, then a value of the natural system to these three areas is $2180 in this example. Thus, we have been able to derive a way of placing a value of the natural system based on the vital function it is providing for man! Interestingly, it is related to the notion of consumer surplus in economic theory.

This ability to place a value on the natural system is of vital importance in resource management. Traditionally, we have attempted to justify open space retention based on its recreation and esthetic values. This has not provided convincing justification for preserving many of these areas. Using this technique one can prove, from a functional basis, that these areas are providing other valuable services for man.

Implementation Problems

Unfortunately, the problem is not yet solved. What happens if we attempt to implement the solution that charges only the actual cost but retains the optimal solution? If we do so, then areas 1 and 3 will probably object

since area 2 derives most of the savings. To avoid this possibility, one might charge the market price, $c + t_i + \pi$. If this is done, then the consumer surplus is as shown below:

Area	Consumer Surplus if $(c + t_i + \pi)$ is Charged
1	2(100) = \$ 200
2	0(270) = 0
3	6(50) = 300
Net Revenue to "Off-Site Facility"	4(420) = 1680
	\$2180

This solution has at least two problems: (1) what to do with the net revenue, and (2) the fact that any positive incentive to area 2 to participate has been eliminated since he is now indifferent between on-site control and off-site control.

The above problem is amenable to attack using N-person game theory. The concepts involved will be presented in Chapter 7. The efficiency and equity questions discussed above also arise in urban wastewater management problems involving several groups/purposes. This is discussed in the next section.

3.5 Multipurpose-Multigroup Planning

Traditionally, cities have utilized a single-purpose, single-group planning approach in the area of water quality

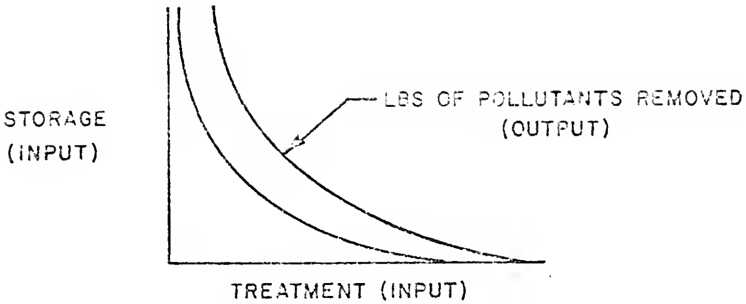
management wherein each purpose is accomplished independent of the other purposes as well as of other cities within the area. However, multipurpose as well as multigroup planning may be more cost effective for area-wide waste treatment management. Some of the concepts involved in multipurpose and multigroup planning are outlined below.

Current technology for wastewater control is based on the concepts of two inputs and one output. The isoquants for the three purposes of area-wide waste treatment management are illustrated in Figure 3-13.

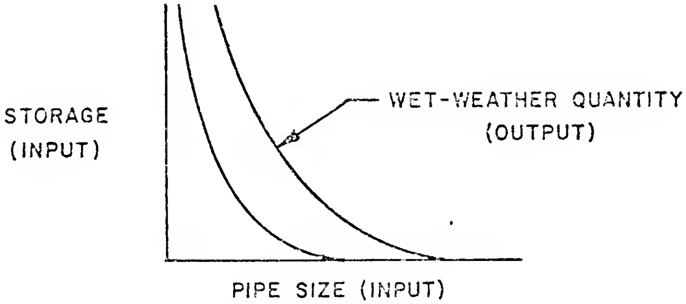
Traditionally, dry-weather quality control is accomplished through treatment. This requires designing these facilities so that they can accommodate the infrequently occurring peaks. Recently it has been advocated that a combination of storage and treatment may be more cost effective (American Society of Civil Engineers, 1975).

Wet-weather quality control can be accomplished either by providing pipes with adequate capacity to provide required drainage or by providing storage facilities in conjunction with smaller pipes. Traditionally, drainage is usually accomplished by installing pipes only. However, the limitation on the amount of runoff that can be discharged such as stated in the previous chapter may require a combination of the two inputs.

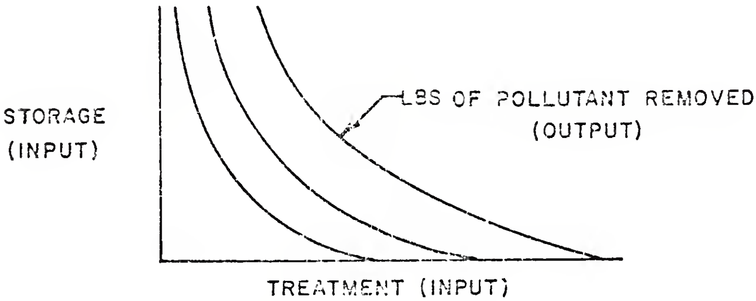
The inputs and outputs for wet-weather quality control are similar to those for dry-weather quality control.



(a) DRY-WEATHER QUALITY CONTROL



(b) WET-WEATHER QUANTITY CONTROL



(c) WET-WEATHER QUALITY CONTROL

Figure 3-13. Relationship Between Inputs and Outputs—
Urban Wastewater Management

However, because of the dynamic nature of stormwater flow and quality, a combination of storage and treatment will usually be more economical.

If planning for each purpose is accomplished separately, it would involve treatment facilities for dry-weather treatment, storage for stormwater quantity control and storage and treatment for wet-weather quality control. By combining stormwater quantity and quality control, storage required for quality control can also be utilized for quantity control, thereby reducing the storage requirements for quantity control. Similarly, by combining dry-weather quality control and wet-weather quality control, fuller utilization of the dry-weather treatment facility can be accomplished. Joint dry-wet weather treatment is technically feasible as the pollutants are compatible. This results in less volume of flow and quantities of pollutants to be handled by the wet-weather plant thereby reducing the storage and treatment requirements for wet-weather quality control. Further, during dry-weather periods, wet-weather facilities could be utilized for providing a higher degree of treatment to dry-weather flow. Thus, the objective in multipurpose planning is to take advantage of the complementarities that exist among the three purposes.

Multigroup planning involves various groups or cities within the study area. For example, in formulating

strategies for dry-weather quality control, each city within the planning area may treat its wastewater on-site or join with one or more of the other cities for the purpose of joint treatment. This feature allows the cities to take advantage of the economies of scale in wastewater treatment and transportation and/or less restrictive waste discharge requirements of some locations. Similar concepts could be applied for accomplishing the other two purposes by various cities.

Thus, the economic efficiency objective in multipurpose and multigroup planning may be viewed as reduction in costs of control. This may be viewed as the efficiency objective.

Let $F_j(X)$ = resource cost to purpose or group j;

X = vector of inputs;

y_j = BOD_5 discharge desired by purpose or group j;

$\overline{y}_j$ = BOD_5 discharge limit for purpose or group j; and

$g_{ij}(X,y_j)$ = constraint set for purpose or group j which is comprised of i=1,2,....,m inequalities.

Then, the efficiency problem for the single purpose or group case may be stated as follows:

$$\text{minimize } Z = (F_j(X);\ y_j)$$

$$\text{subject to } g_{ij}\ (X,y_j) \leq 0 \qquad \forall i \qquad (3.16)$$

$$X, y_j \geq 0$$

Using multiobjective solution techniques, the above problem can be written as a single objective optimization problem

$$\begin{aligned} &\text{minimize } Z = F_j(X) \\ &\text{subject to } g_{ij}(X,y_j) \le 0 \qquad i=1,\ldots,m \\ &\qquad\qquad y_j \le \bar{y}_j \\ &\qquad\qquad x,y_j \ge 0 \end{aligned} \tag{3.17}$$

For the multipurpose or multigroup case, the mathematical formulation of the efficiency problem with p purposes or groups is as follows:

$$\begin{aligned} &\text{minimize } Z = (F_1(X);\ y_1),\ (F_2(X);\ y_2), \\ &\qquad\qquad \ldots\ldots,(F_p(X);\ y_p) \\ &\text{subject to } g_{ij}(X,y_j) \le 0 \qquad i=1,\ldots,m \\ &\qquad\qquad\qquad\qquad\qquad\quad j=1,\ldots,p \\ &\qquad\qquad X,y_j \ge 0 \end{aligned} \tag{3.18}$$

which can be rewritten as follows:

$$\begin{aligned} &\text{minimize } Z = \sum_{j=1}^{p} F_j(X) \\ &\text{subject to } g_{ij}(X,y_j) \le 0 \qquad i=1,\ldots,m \\ &\qquad\qquad\qquad\qquad\qquad\quad j=1,\ldots,p \\ &\qquad\qquad \sum_{j=1}^{p} y_j \le \sum_{j=1}^{p} \bar{y}_j \\ &\qquad\qquad X,y_j \ge 0 \end{aligned} \tag{3.19}$$

Thus, while the single purpose or group problem minimizes the cost associated with each purpose or group, the multipurpose or group problem minimizes the overall cost.

In addition to the minimization of overall costs, the objective of each group or purpose is the minimization of the costs assigned to it. This may be viewed as the equity objective. A general formulation of the problem incorporating both efficiency and equity objectives for p purposes or groups may be stated as follows:

$$\text{minimize } Z = (F_1(X);\ y_1),\ (F_2(X);\ y_2), \ldots,\ (F_p(X);\ y_p),\ \left(\sum_{j=1}^{p} F_j(X);\ \sum_{j=1}^{p} y_j\right)$$

$$\begin{aligned} \text{subject to } g_{ij}(X,y_j) &\leq 0 \qquad \begin{array}{l} i=1,\ldots,m \\ j=1,\ldots,p \end{array} \\ X, y_j &\geq 0 \end{aligned} \qquad (3.20)$$

The last term in the objective function represents the regional objective and other terms are as defined previously. Since the performance criteria, Y, are prespecified, the problem reduces to

$$\text{minimize } Z = F_1(X),\ F_2(X), \ldots, F_p(X),\ \sum_{j=1}^{p} F_j(X)$$

$$\text{subject to } g_{ij}(X,y_j) \leq 0 \qquad \begin{array}{l} i=1,\ldots,m \\ j=1,\ldots,p \end{array} \qquad (3.21)$$

continued

$$y_j \le \bar{y}_j$$

$$\sum_{j=1}^{p} y_j \le \sum_{j=1}^{p} \bar{y}_j$$

$$X, y_j \ge 0$$

Thus, the efficiency and equity problems may be viewed as a multiobjective problem involving p groups or purposes as well as the regional agency.

3.6 Summary

The discussion presented in this chapter shows that urban wastewater management is a multiobjective as well as multigroup and multipurpose problem. Many strategies can be formulated for solving such a problem. Each strategy may have associated environmental and social impacts. If it is assumed that the control criteria reflect these impacts, then the least-cost strategy may also be viewed as a cost-effective solution.

Environmental effects of an alternative include impacts on hydrology, biology, water quality, air quality, land and resource use. Social impacts include changes in economic activity, employment and dislocation of individuals, businesses, and public services and esthetics. Specification of control criteria does not usually include consideration of all these factors. Therefore, two alternatives

which meet the same control criteria may have different social and environmental impacts. Thus, a least-cost solution may not necessarily be a cost-effective solution. Therefore, it becomes necessary from the point of view of cost effectiveness to determine not only the efficient solution but also the costs of other strategies. Then it is possible to rank these strategies for the purpose of preliminary screening. The remaining alternatives can then be subjected to a detailed evaluation through the preparation of environmental impact statements in order to identify the cost-effective solution. Where the regional solution involves joint utilization of facilities among purposes and/or groups, the question of equity must still be resolved in order to ensure the implementation of the selected solution. In succeeding chapters of this work,procedures for single purposes as well as multipurpose analysis will be presented. The procedures are designed to determine the resource costs associated with various strategies and tradeoffs between different strategies. Procedures for equitable financial arrangements are also discussed. Discussion in the next chapter deals with strategies for domestic wastewater management. In subsequent chapters, resources costs will be expressed as annual costs.

CHAPTER 4

DOMESTIC WASTEWATER MANAGEMENT

4.1 Problem Definition

Domestic wastewater management, in the context of 208 planning, involves determining a cost-effective regional solution for treatment and disposal of dry-weather sewage flows generated within a planning area subject to meeting the control criteria established by the Basin Plan. This effort entails an evaluation of the resource costs of the least-cost strategy as well as other regional wastewater management strategies in order for the decision maker to determine tradeoffs between resource costs and environmental and social values associated with different strategies. Three important factors which form the basis for the regionalization of domestic wastewater management are: (1) the economies of scale in wastewater treatment and transmission; (2) less restrictive waste treatment requirements elsewhere; and/or (3) excess treatment capacity elsewhere. Thus, determination of the resource costs associated with a domestic wastewater management strategy is basically a treatment versus pipeline tradeoff problem.

During the last decade several mathematical models have been formulated for addressing regional wastewater management problems. Deininger and Loucks (1972) have presented an excellent review of these models. All of these models assume a number of wastewater sources and possible treatment sites. However, one category of these models assumes a uniform level of treatment at all possible locations and evaluates the tradeoff between treatment plants and pipelines. The other category is designed to maintain a minimum of dissolved oxygen in the receiving water and permits such options as variable degree of treatment at different locations, low flow augmentation and instream aeration. The latter category takes advantage of the assimilative capacity of the receiving water body in order to reduce the cost of waste treatment. The objective in all of the above optimization models is the determination of the least-cost solution for the entire region. When a regional wastewater management problem involves a large number of wastewater sources and treatment sites, the number of pipeline-treatment combinations becomes very large. The economies of scale in wastewater treatment and transportation add nonlinearities to the problems. Due to these reasons many investigators have proposed special algorithms to solve their particular problem and as such these solution techniques have not found wide-spread acceptance and usage.

This chapter presents simple optimization procedures that can be utilized for formulating alternative wastewater management strategies and for evaluating their annual costs. This procedure is simple and easy to use. It does not guarantee, in general, that the solution is the regional optimum. However, it should provide a very close approximation. The level of accuracy is compatible with other screening models to be presented later.

4.2 Strategies for Dry-Weather Quality Control

Domestic wastewater management primarily involves the removal of pollutants such as BOD_5, SS, coliforms, etc. Given the total pollutant loads in the raw wastewater and the waste load allocation or the control criteria, the level of waste treatment as well as the size of the required treatment facility can be determined. The annual costs for dry-weather quality control can then be determined as follows:

Let $\hat{D}$ = required design capacity of the treatment facility, mgd;

D = average annual wastewater flow, mgd ($D \leq \hat{D}$); and

L = distance between the source and the treatment facility, miles.

The annual costs for dry-weather quality control

= annual treatment costs + annual transmission costs.

The annual treatment costs are comprised of amortized capital costs plus operation and maintenance costs.

Let TC^{ψ} = total treatment costs, dollars per year, associated with treatment level, ψ;

CA^{ψ} = amortized capital costs, dollars per year, associated with treatment level, ψ;

OM^{ψ} = operation and maintenance costs, dollars per year, associated with treatment level, ψ; and

ψ = 1 (primary), 2 (secondary), or 3 (tertiary).

Then,

$$TC^{\psi} = CA^{\psi} + OM^{\psi} \tag{4.1}$$

According to the cost functions presented in Table 2-5,

$$CA^{\psi} = l_{\psi}\hat{D}^{m\psi} \tag{4.2}$$

$$OM^{\psi} = p_{\psi}D^{q\psi} \tag{4.3}$$

where l and p are the coefficients and m and q are the exponents in the cost functions for treatment level, ψ.

The total annual cost of wastewater transmission (dollars per year) as listed in Table 2-5 is

$$= 11{,}900\ D^{0.51}L \tag{4.4}$$

Thus, the total treatment plus pipeline cost, C, is

$$C = CA^{\psi} + OM^{\psi} + 11{,}900\ D^{0.51}L \tag{4.5}$$

A typical planning area usually involves many wastewater sources and treatment plant sites. Each source may have the option of treating its wastewater onsite or piping

it to another location for treatment. From a regional standpoint, it is necessary to determine the annual costs associated with various regional strategies.

Smith (1971) suggested the break-even pipe length approach for determining the economically feasible combinations. He considered two cases: (1) the economies of scale in treatment due to consolidation; and (2) the use of available excess capacity. His approach, which is limited to two sources, consists of determining the savings in treatment costs due to consolidation and using these savings to determine the maximum distance the two sources can be apart in order for the joint treatment to be economically feasible. Smith suggested the use of break-even pipe length tables for determining the economically feasible combinations. In the next section, Smith's approach is generalized so that it is applicable to various cases that may arise in area-wide domestic wastewater management.

4.3 Optimization Procedure

Consider the case involving two sources, i and j. Assume that the two alternative domestic wastewater management strategies are: (1) each source treats its wastewater on-site; or (2) a joint treatment facility is located at source j.

Let $\hat{D}_i$ = design capacity of the treatment facility required to serve source i, mgd;

D_i = annual average wastewater flow originating at source i, mgd;

L_{ij} = pipeline distance between source i and source j;

ψ = level of treatment required for a single facility;

ϕ = level of treatment required for a joint facility;

C_i^{ψ} = annual costs for treatment facility located at source i, dollars per year;

C_{ij}^{ϕ} = annual cost for joint facility located at source j, dollars per year;

CA_i^{ψ} = amortized capital costs for treatment facility located at source i; and

OM_i^{ψ} = operation and maintenance costs for treatment facility located at source i, dollars per year.

For on-site treatment, wastewater transmission costs are zero. In accordance with equation (4.1),

$$C_i^{\psi} = CA_i^{\psi} + OM_i^{\psi} = 1_{\psi}\hat{D}_i^{m_{\psi}} + p_{\psi}D_i^{q_{\psi}} \tag{4.6}$$

and

$$C_j^{\phi} = CA_j^{\phi} + OM_j^{\phi} = 1_{\phi}\hat{D}_j^{m_{\phi}} + p_{\phi}D_j^{q_{\phi}} \tag{4.7}$$

The annual costs associated with on-site control strategy are $C_i^{\psi} + C_j^{\psi}$. The annual costs associated with a joint treatment strategy are

$$C_{ij}^{\phi} = 1_{\phi}(\hat{D}_i + \hat{D}_j)^{m_{\phi}} + p_{\phi}(D_i + D_j)^{q_{\phi}} + 11{,}900\ D_i^{0.51}L_{ij} \tag{4.8}$$

Thus, pipeline cost cannot exceed the savings from joint treatment, i.e.,

$$\text{savings} = C_i^{\psi} + C_j^{\psi} - C_{ij}^{\phi} \tag{4.9}$$

Thus, the break-even pipe length, L^*_{ij}, is

$$L^*_{ij} = \frac{C_i^{\psi} + C_j^{\psi} - C_{ij}^{\phi}}{11{,}900\ D_i^{0.51}} \tag{4.10}$$

Given the wastewater loads and the required level of treatment, the economic feasibility of joint treatment can be determined by comparing the actual pipeline distance between the two sources with the maximum pipeline distance that these sources can be apart. Application of equation (4.10) to various cases that arise in area-wide wastewater planning is presented below:

Case 1: Pipe effluent to site j to reduce treatment level at site i from ψ to ψ'

$$L^*_{ij} = \frac{C_i^{\psi} - C_i^{\psi'}}{11{,}900\ D_i^{0.51}}$$

Case 2: Pipe wastewater from i to j. No plant exists at present. L^*_{ij} can be found from equation (4.10).

Case 3: Pipe wastewater from i to j. Existing plant at j has excess capacity. City i pays incremental operation and maintenance costs. In this case,

$$L^*_{ij} = \frac{C_i^{\psi} - p_{\phi}(D_i + D_j)^{q_{\phi}} + p_{\phi}(D_j)^{q_{\phi}}}{11{,}900\ D_i^{0.51}}$$

The above results can be extrapolated to more than two sites by simply redefining the $\overline{ij}$ pair as a single source and comparing the new source with another site k. Note that an $\overline{ij}$ pair may be infeasible by itself, but becomes feasible when combined with another site k. This procedure lacks complete generality in that the partitioning of the entire set of sites into pairs is not unique. The resulting combinatorial problems make overall optimization unwieldy. Fortunately, the analyst's knowledge about the area and engineering judgement should permit him to derive a good solution based on this simple procedure. Exact solutions can be obtained for small n (e.g., $n < 10$) as shown in the illustrative example to be presented next.

4.4 Illustrative Example

The hypothetical planning area shown in Figure 2-2 has seven wastewater sources. Each source may treat its wastewater onsite or pipe it to another source for joint treatment. Thus, under assumed conditions, there are seven possible treatment sites. The control criteria for dry-weather control listed in Table 2-9 require secondary treatment or level 2 at all locations. The incentive for regionalization in this case is the economies of scale.

Feasible piping/treatment combinations are shown in Figure 4-1. If it is assumed that no source may pipe its wastewater for treatment at an upstream location and

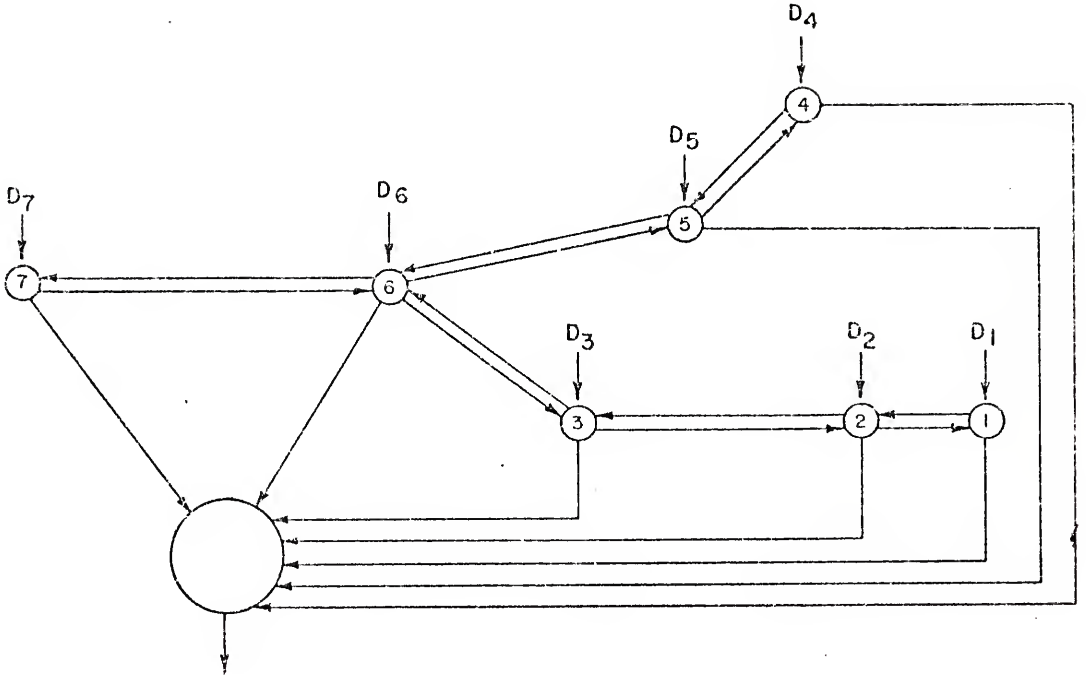


Figure 4-1. Permitted Piping-Treatment Combinations for Domestic Wastewater Management with Upstream Pumping—Hypothetical Planning Area

that the physical and topographic features do not permit wastewater from sources 1, 2, and 3 to be transmitted to either source 4 or 5 and vice versa, the number of piping/treatment combinations are reduced substantially as shown in Figure 4-2. Using equation (4.10) to determine the number of combinations that are economically feasible yields the following matrix:

		Source						
	From \ To	1	2	3	4	5	6	7
Source	1	Y	Y	Y	N	N	N	N
	2	N	Y	Y	N	N	Y	N
	3	N	N	Y	N	N	Y	N
	4	N	N	N	Y	N	Y	N
	5	N	N	N	N	Y	Y	N
	6	N	N	N	N	N	Y	N
	7	N	N	N	N	N	N	N

Y = Yes; N = No

Also, sources 1 and 2 can treat jointly with 3 and the following groups can treat jointly with 6.

Size	Feasible Upstream Combinations
2	$\overline{13}$, $\overline{23}$, $\overline{34}$, $\overline{35}$, $\overline{45}$
3	$\overline{123}$, $\overline{134}$, $\overline{135}$, $\overline{234}$, $\overline{235}$, $\overline{345}$
4	$\overline{1234}$, $\overline{1235}$, $\overline{1345}$, $\overline{2345}$
5	$\overline{12345}$

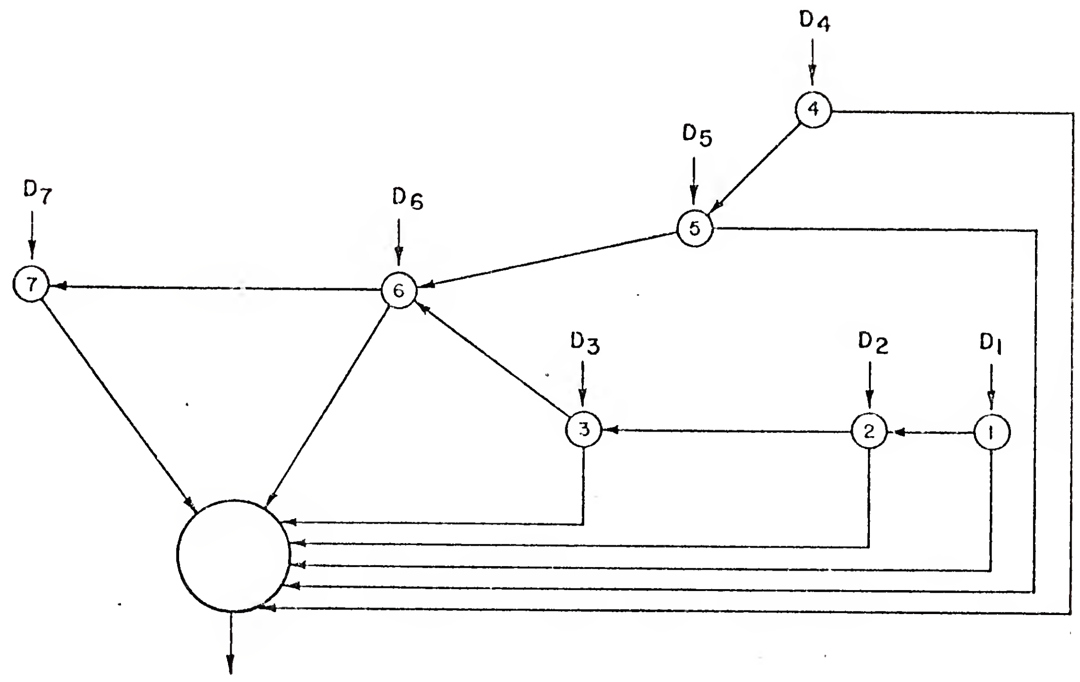


Figure 4-2. Piping-Treatment Combinations for Domestic Wastewater Management Without Upstream Pumping—Hypothetical Planning Area

The above analysis can be utilized to formulate alternative strategies. The annual costs for various strategies can then be computed by using the actual pipeline distances between the sources involved. The annual costs associated with six different strategies including the least-cost solution are listed in Table 4-1.

4.5 Summary

This chapter has presented procedures for formulating strategies for domestic wastewater management and for determining the annual costs associated with these strategies. In order to define a cost-effective solution, these strategies must be evaluated further. If the strategy selected for implementation is complete decentralization, then equity questions do not arise. In all other cases, the facilities will be shared between more than one source and it is necessary to determine equitable cost sharing among those sources. This aspect of the problem will be discussed in Chapter 7. Urban wastewater treatment management involves not only management of domestic wastewater but wet-weather flows as well. Procedures for evaluating the strategies for wet-weather control are presented in the next chapter.

Table 4-1

Annual Costs for Various Strategies for
Domestic Wastewater Management-Hypothetical Planning Area

Strategy Number	Description	Least Cost Solution $\$x10^6/yr$
1	Each City goes Alone	1.072
2	Each Polity goes Alone	1.045
3	Polities 1 and 2 go Together and Polity 3 goes Alone	1.045
4	Polities 2 and 3 go Together and Polity 1 goes Alone	1.026
5	Polities 1 and 3 go Together and Polity 2 goes Alone	0.996
6	All Polities go Together	0.965

CHAPTER 5

STORMWATER MANAGEMENT

5.1 Problem Definition

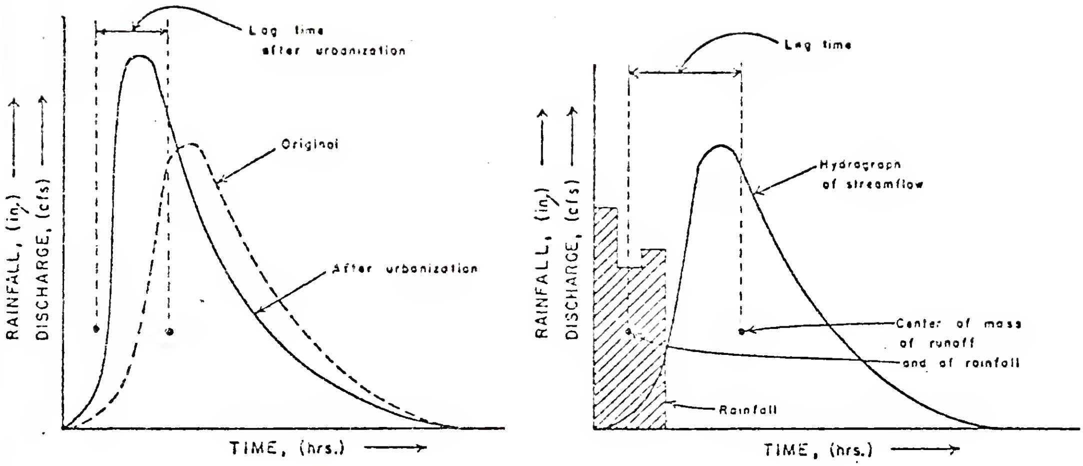
The problem of stormwater management involves control of relatively large flow rates during dynamically variable "wet-weather periods of short duration." Formulation of control strategies requires determination of the quantity and quality of these flows expected to be generated within the planning area. It also requires specification of control criteria and a knowledge of various control devices for attaining that level of control. In Chapter 2, simulation models for predicting the quantity and quality of wet-weather flows were discussed. A brief summary of wet-weather control devices and their costs was also presented in Chapter 2. Given this information, it is necessary to determine the strategies for wet-weather control and the resource costs associated with those strategies.

For domestic wastewater management, the economies of scale in wastewater treatment and/or less restrictive waste treatment requirements at some locations within a

planning area provide incentives for regionalization of facilities. These factors become less important in wet-weather control because of the relatively high flow rates encountered. Unless the flow is transported in existing creeks or drainage ditches for control at another location, economies of scale in control costs may not be sufficient to offset the additional piping costs. Further, a wide variety of on-site control options (structural and non-structural) and differing stormwater quantity goals by each community make regionalization of wet-weather control even less attractive. Thus, in general, wet-weather control must be accomplished individually by each city. Within a planning area, wet-weather control may involve quantity and/or quality control. Therefore, in this chapter, procedures will be presented for formulating alternative wet-weather quantity and quality control strategies and determining the annual costs associated with these strategies.

5.2 Strategies for Wet-Weather Quantity Control

Urbanization results in an increase in the rate as well as the volume of surface runoff as illustrated in Figure 5-1. Factors which contribute to the increased runoff are the paved surfaces such as rooftops, parking lots, streets and the use of storm sewers. The impact of storm sewers is also illustrated in Figure 5-1. The



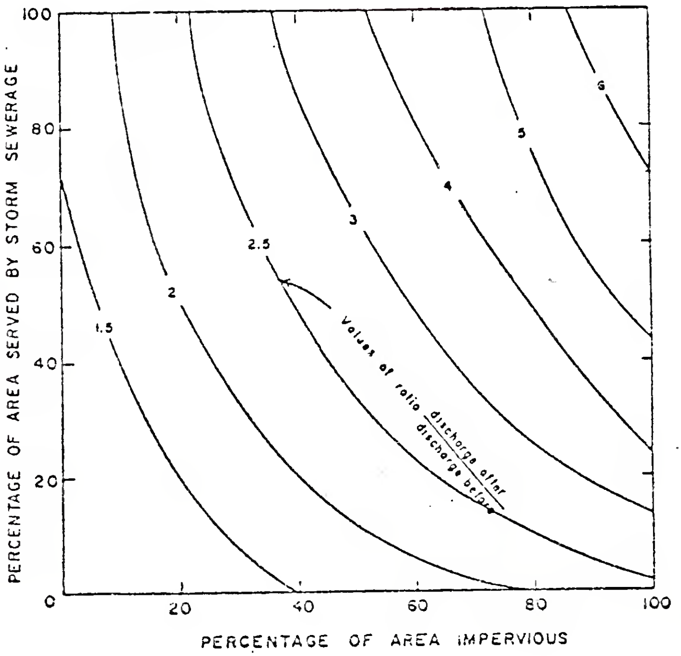


Figure 5-1. Effect of Urbanization on Peak Discharges (Leopold, 1968)

objective of wet-weather quantity control is to reduce or regulate the quantity of stormwater runoff.

A reduction in the quantity of stormwater runoff can be effected by increasing on-site detention capabilities. Current efforts by many cities consist chiefly of an attempt to regulate or limit the rate of surface runoff from urbanized areas to the rates which existed prior to development. The allowable release rate is usually based on a storm with a stated return frequency. In the Chicago metropolitan area, the allowable release rate is set equal to a three-year storm prior to development from a watershed with an overall coefficient of 0.15 (MSDGC, 1970). In the State of Maryland, the allowable release rate is set equal to the release rate of a two-year storm prior to development for a watershed having an overall runoff coefficient of 0.20 (Montgomery Soil Conservation District, 1971).

Compliance with such quantity control criteria requires construction of on-site/off-site storage facilities. On-site storage may consist of parking lot and rooftop storage, surface or underground tanks, tunnels and infiltration basins and trenches. Off-site storage may consist of stream channel storage and/or surface and underground tanks. The objective is to meet the specified criteria at minimum cost. Where a variety of on-site and off-site control alternatives are available, the problem becomes

one of selecting the optimal mix. In order to accomplish this objective, it is necessary to estimate the amount of storage that must be provided and to inventory the various control alternatives and their associated cost. The volume of storage needed is dependent upon the allowable release rate. Simulation models such as the SWMM can be utilized to generate hydrographs for developed and undeveloped conditions. The allowable release rate can be determined based on undeveloped conditions. Knowing this, the required storage volume can be computed from the hydrograph of developed conditions.

Let V = amount of storage required;

$\overline{W}$ = allowable release rate; and

RO = runoff rate under developed conditions.

Then

$$V = \text{maximum value of } \sum_{\text{time}} (RO-\overline{W})$$

Instead of using the simulation model, a less accurate but more common method of estimating the release rate and the runoff rate under developed conditions is to use the rational formula

$$RO = GBIA \tag{5.1}$$

where RO = runoff, cubic feet per second (cfs);

I = intensity of rainfall, inches per hour;

B = runoff coefficient;

A = area in acres; and

$$G = \text{conversion factor} = 1 = \frac{\text{cfs}}{\text{acre-in/hr}} .$$

The rational method is relatively easy to use as all of the required information with the exception of the runoff coefficient, B, is readily available. The runoff coefficient is a function of the amount of impervious area and can be computed as follows (Hydrologic Engineering Center, 1975):

$$B = 0.15\left(\frac{100-H}{100}\right) + .9H \tag{5.2}$$

where H = percent of the total area that is impervious.

According to Graham et al. (1974)

$$H = 91.32 - 69.34(0.9309)^{PD} \tag{5.3}$$

where PD = population density in the developed area under consideration.

When a planning area involves several cities needing wet-weather quantity control, it is possible to formulate this problem as an optimization problem. This is illustrated in the next section by means of the hypothetical planning area example.

5.3 Wet-Weather Quantity Control Optimization

Consider the hypothetical planning area shown in Figure 2-2. The wet-weather quantity control criteria listed in Table 2-9 require that cities 1, 2 and 3 restrict

the amount of runoff generated under developed (1990) conditions to what is occurring at present (1975). The quantity control criterion is designed to limit the flow discharged from the creek into the main river to what is occurring at present. Data required for determining the volume required for quantity control are presented in Table 5-1. The computations required for calculating the storage volume for each of the three cities are presented in Table 5-2. These computations are based on using the rational method. The specified control criteria can be met in one of two ways. Each city may provide the required storage volume independently by utilizing its on-site control options or a mix of on-side and off-site control options may be utilized by all three cities acting jointly. Assume that the off-site control option consists of constructing instream storage just above the confluence of the creek with the main river. Then each city has the option of discharging its stormwater into the creek or controlling it on-site.

Let V_i = volume of storage required for city i;

x_{ij} = volume controlled by the j^{th} control option for city i, where j = control option;

$\bar{x}_{ij}$ = maximum available for control option j in city i;

c_{ij} = amortized unit cost of j^{th} control option for city i, annual dollars per acre-feet;

t_i = amortized unit cost of stormwater transmission from city i to control facility;

Table 5-1

Data for Wet-Weather Quantity Control—
Hypothetical Planning Area

	City 1	City 2	City 3
Developed Area (1975-1990), ac	200.00	160.00	600.00
Natural Runoff Coefficient, B_u	0.15	0.15	0.15
Time of Concentration, min	45.00	45.00	45.00
Intensity of Rainfall, in/hr, I	3.00	3.00	3.00
Allowable Release Rate,[a] $\overline{W}$, ft^3/sec	90.00	72.00	270.00
Population (1975-1990)	1000.00	1000.00	6000.00
Population Density in Developed Area,[b] PD, persons/ac	5.00	6.70	10.00
Percent Imperviousness in Developed Area,[b] H	42.85	48.40	57.43
Runoff Coefficient in Developed Area,[c] B_D	0.47	0.51	0.58

[a]From equation (5.1).

[b]From equation (5.3).

[c]From equation (5.2).

Table 5-2

Computations of Required Storage Volumes for Wet-Weather Quantity Control
Hypothetical Planning Area

Rainfall Duration		Intensity of Rainfall in/hr under 10-year Storm, I	City 1			City 2			City 3		
Hours	Minutes		Runoff Rate ft^3/sec = B_0IA	Stored Rate ft^3/sec =(Runoff Rate− Release Rate)	Storage Volume ac-ft = Stored Rate $\frac{\times \text{ time in hrs}}{12}$	Runoff Rate ft^3/sec = B_0IA	Stored Rate ft^3/sec =(Runoff Rate− Release Rate)	Storage Volume ac-ft = Stored Rate $\frac{\times \text{ time in hrs}}{12}$	Runoff Rate ft^3/sec = B_0IA	Stored Rate ft^3/sec =(Runoff Rate− Release Rate)	Storage Volume ac-ft = Stored Rate $\frac{\times \text{ time in hrs}}{12}$
0.17	10	6.8	639.2	549.2	7.78	554.9	482.9	6.84	2366.4	2096.4	29.70
0.33	20	5.0	470.0	380.0	10.45	408.0	336.0	9.24	1740.0	1470.0	40.43
0.50	30	4.0	376.0	286.0	11.92	326.4	254.4	10.60	1392.0	1122.0	46.75
1.0	60	2.6	244.4	154.4	12.83 max	212.2	140.2	11.68 max	904.8	634.8	52.90 max
1.5	90	1.95	183.3	93.3	11.66	159.1	87.1	10.88	678.6	408.6	51.08
2.0	120	1.55	145.7	55.7	9.28	126.5	54.5	9.08	539.4	269.4	44.90

Q_i = volume of flow from city i to central facility;

$\overline{V}$ = maximum amount of storage available at off-site location (instream storage); and

c = amortized unit cost of off-site storage at central control.

The optimization problem, in linear programming form, shown in Figure 5-2, is stated mathematically as follows:

$$\text{minimize} \quad \sum_{i=1}^{3} \sum_{j=1}^{4} c_{ij}x_{ij} + \sum_{i=2}^{3} t_i Q_i + c\overline{V} \qquad (5.4)$$

subject to

$$\sum_{j=1}^{4} x_{ij} + Q_i = V_i \qquad \forall i$$

$$\sum_{i=1}^{3} Q_i \leq \overline{V}$$

$$x_{ij} \leq \overline{x}_{ij} \qquad \forall i,j$$

$$x_{ij} \geq 0 \qquad \forall i,j$$

$$Q_i \geq 0 \qquad \forall i$$

The cost of decentralization strategy, i.e., on-site control, is

Z_1 = annual cost of on-site control to study area 1 = \$39,000 per year;

Z_2 = annual cost of on-site control to study area 2 = \$20,000 per year; and

Z_3 = annual cost of on-site control to study area 3 = \$189,000 per year.

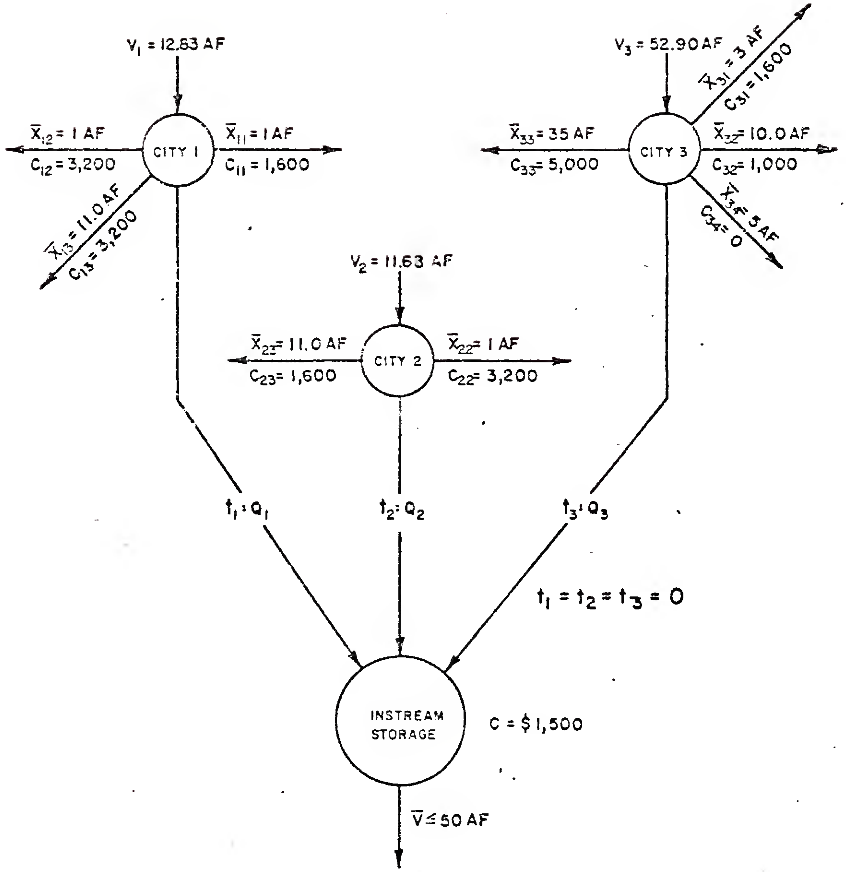


Figure 5-2. Network Representation of Wet-Weather Quantity Control Problem—Hypothetical Planning Area

and $Z = Z_1 + Z_2 + Z_3$ = \$248,000 per year.

The above problem can be solved by linear programming techniques. In this case, various strategies are on-site control or a mix of on-site controls in conjunction with off-site control. By setting $Q_i = 0$ the annual costs of on-site control are obtained. By assuming different values of $\overline{V}$, annual costs associated with a mix of on-site and off-site controls associated with that strategy are determined (see Table 5-3).

5.4 Strategies for Wet-Weather Quality Control

Because of the intermittency and extreme variability of wet-weather flow and its associated pollutants, both in time and space, there is no analogous "average" design condition for stormwater quality control as compared to dry-weather quality control. Therefore, if wet-weather quality control is accomplished through treatment alone, the device must be capable of handling peak flows. Depending upon the design conditions, these peaks occur infrequently and therefore the full capacity of the treatment facility is seldom utilized. If, on the other hand, the treatment facility is designed for more frequently occurring storms, the design capacity is exceeded, often resulting in the discharge of untreated flows. In general, some combination of storage and treatment would be more

Table 5-3

Annual Costs for Wet-Weather Quantity Control
Hypothetical Planning Area

Strategy	Description	Total Annual Cost $\$ \times 10^3/yr$
1	On-site Control only	248
2	On-site Control with 10 ac-ft of Off-site Control	213
3	On-site Control with 20 ac-ft of Off-site Control	178
4	On-site Control with 30 ac-ft of Off-site Control	143
5	On-site Control with 40 ac-ft of Off-site Control	118
6	On-site Control with 50 ac-ft of Off-site Control	105

cost effective for wet-weather quality control as illustrated in Figure 5-3. Storage results in flow equalization and consequently a smaller treatment capacity is required. The concept is further illustrated in Figure 5-4, which shows how storage costs decrease as the treatment rate is increased. The optimal combination of storage and treatment is indicated where total costs are minimum.

In general, there are three ways of specifying wet-weather quality control criteria. These are (1) number of overflows per year, (2) percent annual runoff control, and (3) percent annual pollutant control. Heaney and Huber et al. (1976) show that by defining the interevent time in an appropriate manner, the number of overflow events per year can be converted into the percent annual runoff control. Therefore, given the wet-weather control criteria in terms of percent annual runoff or pollutant control, the objective in wet-weather quality control is to formulate various storage-treatment strategies and to determine the costs associated with these strategies.

Various storage-treatment strategies for wet-weather quality control can be formulated by using the simulation model STORM (Hydrologic Engineer Center, 1975). The system as simulated by STORM is shown in Figure 5-5. For each storage-treatment combination, STORM gives a value of annual percent runoff controlled based on hourly simulation. By making several simulation runs at different storage-treatment

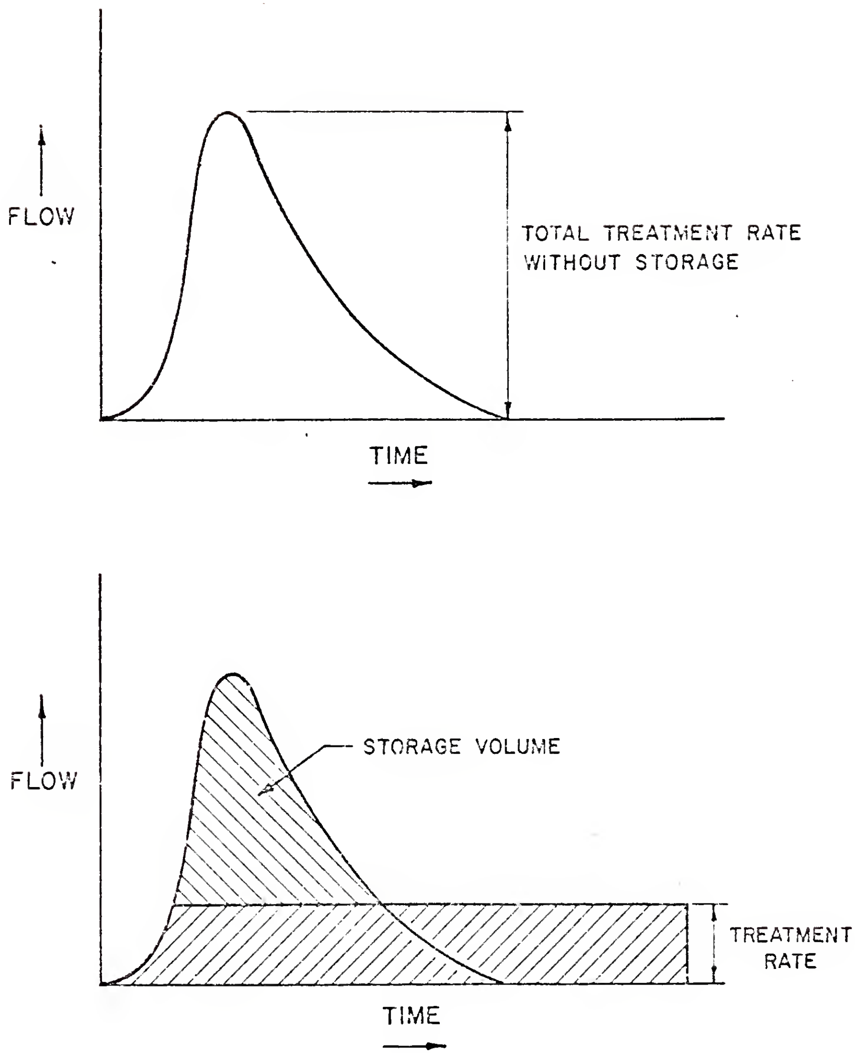


Figure 5-3. Storage Versus Treatment for Wet-Weather Quality Control (Field and Lager, 1974)

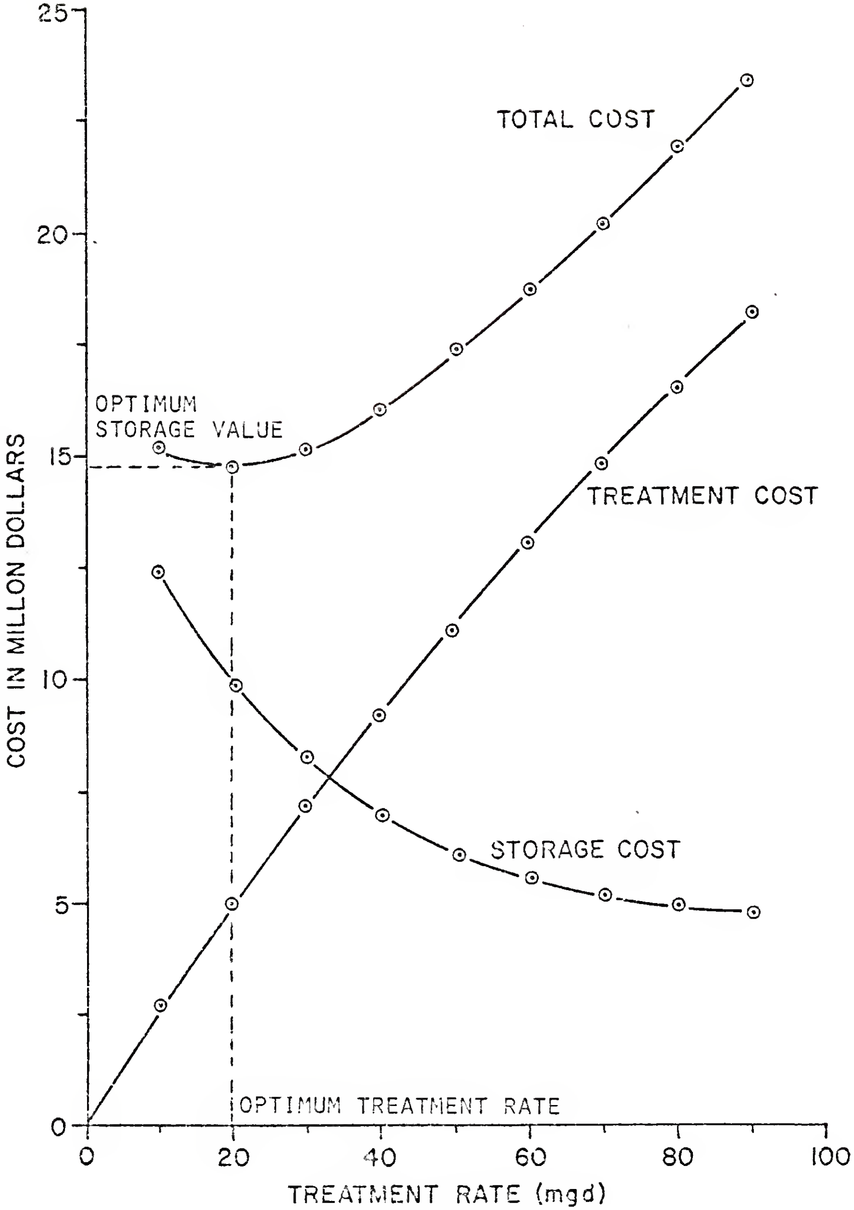


Figure 5-4. Optimal Storage-Treatment for Wet-Weather Quality Control

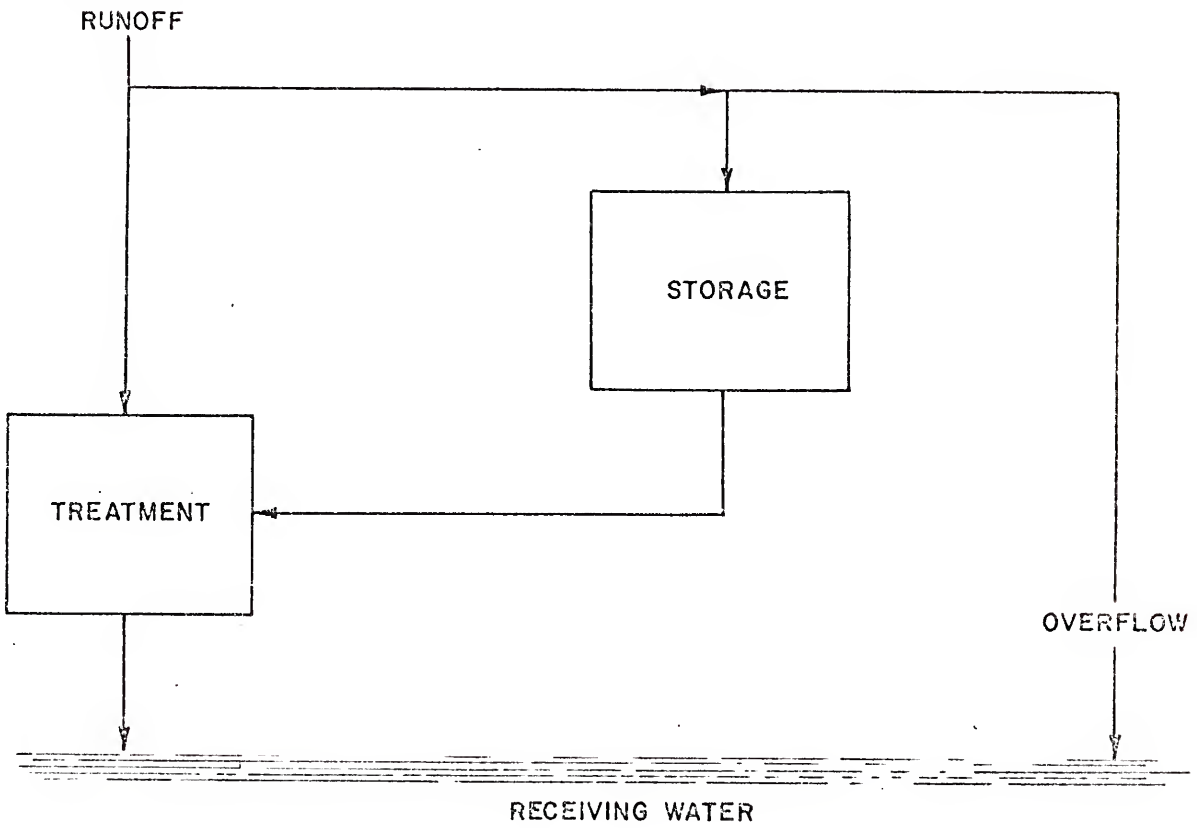


Figure 5-5. STORM Model Simulation of Storage and Treatment for Wet-Weather Quality Control

combinations, respective values for the annual percent runoff controlled can be determined. The results can then be utilized to generate storage-treatment isoquants for various levels of annual runoff control. Murphy (1975) derived such isoquants for five cities, i.e., San Francisco, Denver, Minneapolis, Atlanta, and Washington, D.C. These cities were chosen to represent various regions of the United States as shown in Figure 5-6. The storage-treatment isoquants for Atlanta are shown in Figure 5-7. Given the isoquants and the desired level of annual runoff to be controlled, any point on the isoquant represents a storage-treatment strategy for that level of control. The cost functions presented in Table 2-8 can then be used to determine the cost associated with various storage-treatment strategies. Further, the optimal or least-cost storage-treatment strategy can also be determined graphically by using the iso-cost approach (Murphy, 1975).

The graphical approach for determining the least-cost strategy is relatively easy to use then the storage and treatment costs are linear. However, when they are nonlinear or where the results from a test city within a region are to be extrapolated for approximating the cost of stormwater quality control for other urbanized areas within a region, analytical approach is more useful. The total cost of stormwater quality control may be represented by

URBANIZED AREAS: 1970

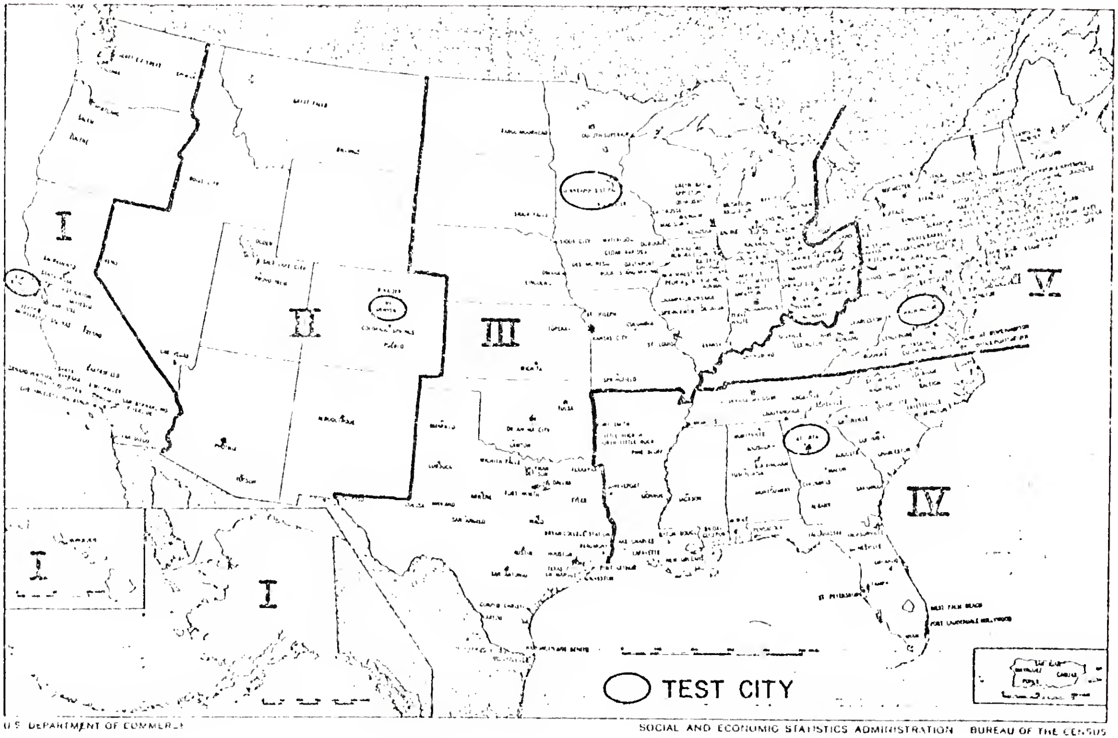


Figure 5-6. Regional Boundaries for Nation-Wide Assessment (Heaney *et al.*, 1976)

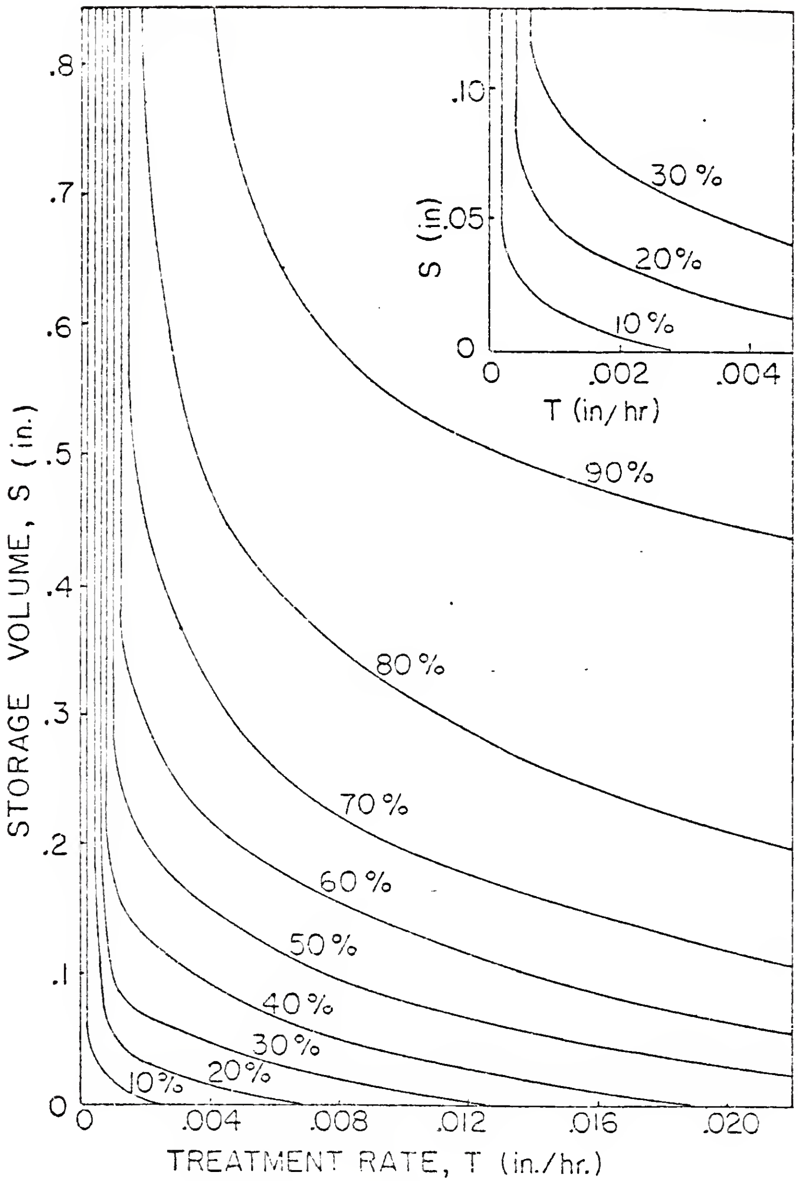


Figure 5-7. Storage-Treatment Isoquants for Various Runoff Control Levels—Region IV: Atlanta (Murphy, 1975)

$$Z = c_S(S) + c_T(T) \qquad (5.5)$$

The relationship between storage, treatment and the percent annual runoff control may be represented by

$$g(R;S,T) = 0 \qquad (5.6)$$

where Z = total cost of stormwater quality control;

$c_S(S)$ = storage costs;

$c_T(T)$ = treatment costs;

S = storage volume;

T = treatment rate; and

R = percent runoff control.

In accordance with the information presented in Table 2-8, equation (5.5) may be written as

$$Z = w_1 \; T^{z_1} + w_2 \; S^{z_2} \qquad (5.7)$$

where T = treatment rate, inches per hour;

S = storage volume, inches;

w_1 & w_2 = cost efficients for treatment and storage respectively, dollars; and

z_1 and z_2 = exponents.

The storage treatment isoquants shown in Figure 5-7 are of the form

$$T = T_1 + (T_2 - T_1)e^{-KS} \qquad (5.8)$$

where T = wet-weather treatment rate, inches per hour;

T_1 = treatment rate at which an isoquant becomes asymptotic to the ordinate, inches per hour;

T_2 = treatment rate at which isoquant intersects the abcissa, inches per hour;

S = storage volume inches; and

K = constant, inch^{-1}.

The value of T_1 occurs at a relatively high storage capacity in combination with a low treatment rate such that the treatment plant operates continuously. T_1 can be found as follows:

$$T_1 = \frac{AR}{8760} \left(\frac{R}{100}\right) = aR \qquad (5.9)$$

where AR = annual runoff, inches per year; and

R = percent runoff control.

By relating the parameters T_1, $T_2 - T_1$, and K to the level of runoff control R, equation (5.5) was fitted to the isoquants derived for all five cities. The $T_2 - T_1$ and K terms versus R were found to be of the following general form:

$$T_2 - T_1 = be^{hR} \qquad (5.10)$$

$$K = de^{-fR} \qquad (5.11)$$

where b, h, d and f are the constants.

Based on this analysis the following equation for the isoquants is obtained.

$$T = aR + be^{hR} - (de^{-fR})S \qquad (5.12)$$

The value of the paramters a, b, h, d and f for various cities are presented in Table 5-4 for storage-treatment isoquants for percent runoff control. The correlation coefficients are also shown in this table. In general the fit is excellent.

Adjusting Isoquants for Treatment Efficiencies and First Flush Effects

The above isoquants equation (5.12) characterizes the percentage of total runoff that passes through "treatment." If the concentration of pollutants is constant and "treatment" efficiency, η, is 1.0 then percent runoff control is synonymous with percent pollutant control. Obviously, this is not the case. Thus, these results need to be refined to account for

(1) treatment efficiency; and

(2) variable concentration due to first flush effects.

Adjustment for Treatment Efficiency

Recall that R is the percent runoff control. Let η equal treatment plant efficiency. If R_1 denotes the

Table 5-4

Values of Parameters and Correlation Coefficients for Isoquant Equations Percent Runoff Control or Percent BOD without First Flush, $\eta = 1.0$ (Malec, 1975)

Region	Test City	a in $hr^{-1}(\% R)^{-1}$	b in hr^{-1}	h $(\% R)^{-1}$	d in^{-1}	f $(\% R)^{-1}$	Correlation $T_2-T_1=be^{hR}$	Coefficient $K=de^{-fR}$
I	San Francisco	0.0000113	0.002604	0.033837	101.090	0.0355920	0.967	0.995
II	Denver	0.0000067	0.001289	0.038214	123.720	0.0308110	0.986	0.989
III	Minneapolis	0.0000125	0.001085	0.055345	211.755	0.0332537	0.988	0.962
IV	Atlanta	0.0000193	0.001836	0.056500	78.610	0.0290515	0.951	0.959
V	Washington, DC	0.0000211	0.002135	0.058828	87.073	0.0345472	0.996	0.973

percent pollutant control, then to realize R_1, one needs to process R_1/η of the runoff. Note that R_1 may be percent BOD removal, percent SS removal, etc. In Table 2-4 representative treatment efficiencies, in terms of BOD_5 removal, were derived for primary and secondary devices. These values are listed below.

Treatment Devices	Assumed Efficiency, η (BOD_5 Removal)
Primary	0.40
Secondary	0.85

Thus, if one desires 25 percent BOD_5 removal with a primary device, then 62.5 percent of the runoff volume must be processed whereas only 29.4 percent of the runoff needs to be processed if a secondary device is selected. Thus, to convert percent runoff control isoquants to percent pollutant control isoquants, one simply uses

$$R = Rl/\eta \qquad (5.13)$$

Adjustment for First Flush

STORM estimates the percent pollutant control as well as percent runoff control. This model has a first flush assumption built into it. Thus, one can estimate the effects of the first flush assumption by generating the percent pollutant control isoquants directly from STORM. This has

been done for percent BOD control for all five cities (Malec, 1975). The results for Atlanta are shown in Figure 5-8. A comparison of a storage-treatment isoquant for percent BOD control with and without first flush is presented in Figure 5-9 for the City of Atlanta. It should be pointed out that for low levels of runoff and BOD control, the difference is not as significant as at higher levels of control.

Equation (5.12) was also fitted to the storage-treatment isoquants for percent BOD control for all five cities. The resulting parameters and correlation coefficients are shown in Table 5-5. The fit is excellent. Thus, one uses the parameter shown in Table 5-5 in conjunction with equation (5.12) instead of those shown in Table 5-4 to account for first flush. Note that the parameters shown in Table 5-5 are based on a treatment efficiency, η, of 1.0. To adjust for the actual treatment efficiency, again one uses equation (5.13). In this case R represents the pollutant control at 100% efficiency.

Asjusting Isoquants for Other Urbanized Areas Within a Region

Within a region, climatological and many other factors needed to generate storage-treatment isoquants are quite similar. Therefore, given the isoquants for a test city within a region, the corresponding isoquants for other cities within that region may be approximated as follows:

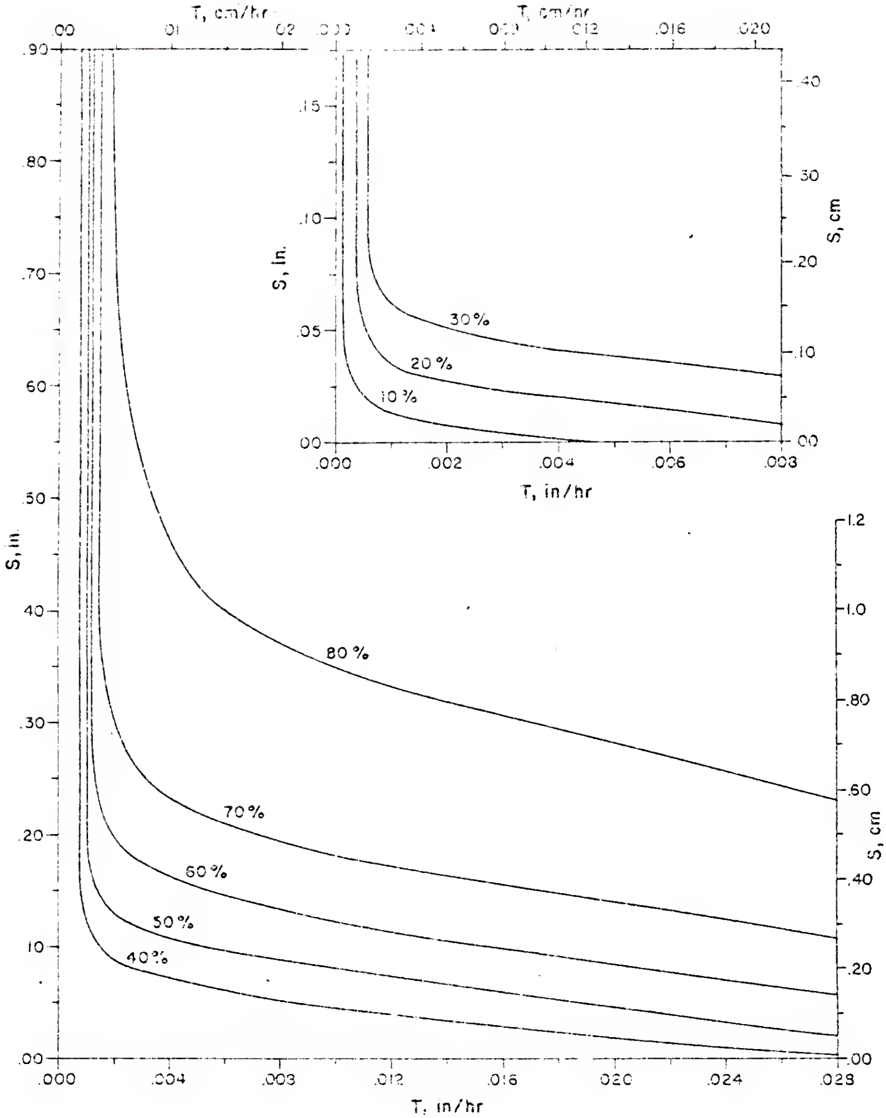


Figure 5-8. Storage-Treatment Isoquants for Various BOD Control Levels—Region IV: Atlanta (Malec, 1975)

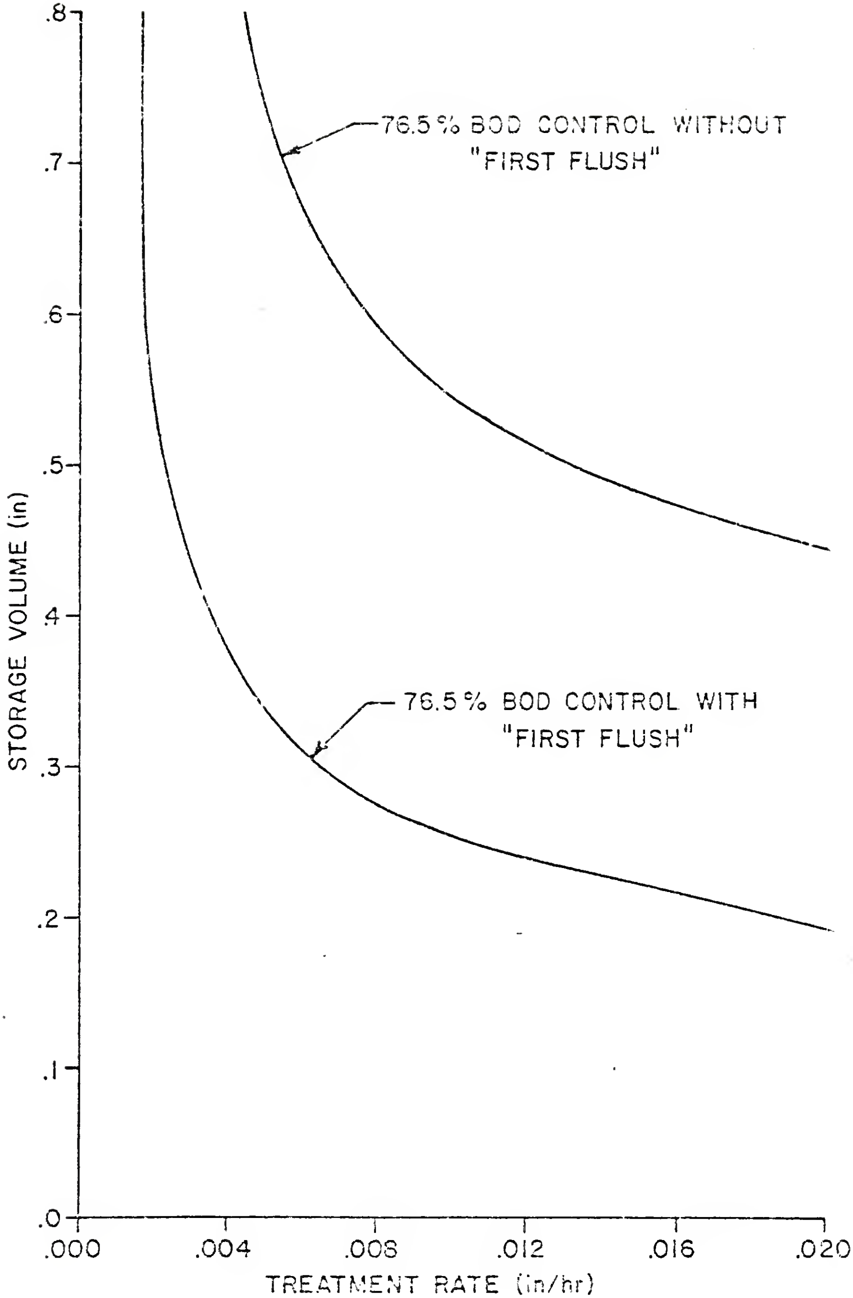


Figure 5-9. Comparison of a Storage-Treatment Isoquant with and without First Flush—Region IV: Atlanta (Malec, 1975)

Table 5-5

Values of Parameters and Correlation Coefficients for Isoquant Equations
Percent BOD Control with First Flush, η = 1.0

Region	Test City	a	b	h	d		Correlation	Coefficient
		in $hr^{-1}(\% R_1)^{-1}$	in hr^{-1}	$(\% R_1)^{-1}$	in^{-1}	$(\% R_1)^{-1}$	$T_2-T_1=be^{hR}$	$K=de^{-fR}$
I	San Francisco	0.0000113	0.0023666	0.0382288	187.2858	0.0328201	0.987	0.987
II	Denver	0.0000067	0.0014865	0.0434124	177.3463	0.0290000	0.996	0.981
III	Minneapolis	0.0000125	0.0015157	0.0480651	213.7156	0.0304724	0.998	0.985
IV	Atlanta	0.0000193	0.0032524	0.044997	174.7269	0.0323855	0.995	0.968
V	Washington, DC	0.0000211	0.0021282	0.0495669	147.6526	0.0323801	0.996	0.955

let AR_i = annual runoff in city i; and

AR_j = annual runoff in test city j (San Francisco, 9.88 inches; Denver, 5.9 inches; Minneapolis, 10.99 inches; Atlanta, 16.93 inches, and Washington, D.C., 18.50 inches (Murphy, 1975)).

Then

$$a_{ij} = AR_{i}/8.76 \times 10^5 \tag{5.14}$$

$$b_{ij} = \left(\frac{AR_i}{AR_j}\right) b_j \tag{5.15}$$

$$h_{ij} = h_j \tag{5.16}$$

$$d_{ij} = \left(\frac{AR_j}{AR_i}\right) d_j \tag{5.17}$$

$$f_{ij} = f_j \tag{5.18}$$

where a_{ij}, b_{ij}, h_{ij}, d_{ij} and f_{ij} are parameters for city i in region j and b_j, h_j, d_j and f_j are the parameters for test city in region j.

Thus, if one knows the annual runoff in the city under consideration, one can approximate the storage-treatment isoquant for any desired level of control with or without first flush. Given the control criteria, one can also formulate alternative storage-treatment strategies. The cost associated with a control strategy can then be

determined from equation (5.7) by using appropriate values of w_1, w_2, z_1 and z_2. The optimization procedure for determining the least-cost strategy is presented in the next section.

5.5 Wet-Weather Quality Control Optimization

The wet-weather quality control optimization problem may be stated as follows:

$$\text{minimize} \quad Z = w_1 T^{z_1} + w_2 S^{z_2} \tag{5.19}$$

subject to

$$T = T_1 + (T_2 - T_1) e^{-KS} \tag{5.20}$$

$$T, S \geq 0 \tag{5.21}$$

where all terms are defined earlier.

For the special case, when storage and treatment costs are linear, i.e., z_1 and $z_2 = 1$, and letting $w_1 = c_T$ and $w_2 = c_S$ gives

$$S^* = \max \begin{cases} \frac{1}{K} \ln \frac{c_T}{c_S} [K(T_2 - T_1)] \\ 0 \end{cases} \tag{5.22}$$

and

$$T^* = T_1 + (T_2 - T_1) e^{-kS^*} \tag{5.23}$$

The optimal value of the objective function is

$$Z^* = c_S S^* + c_T T^* \tag{5.24}$$

The optimization procedure presented above can be utilizied to generate curves showing percent pollutant control versus annual costs for various wet-weather treatment devices listed in Table 2-7. The curves for percent BOD control for Atlanta for primary as well as secondary devices, assuming first flush and that the representative costs for treatment devices are linear, are shown in Figure 5-10. These curves are based on the following costs:

c_S = $122e^{0.16PD}$ dollars per acre-inch

c_T = \$2610/acre-inch per hour for a primary device

c_T = \$9800/acre-inch per hour for a secondary device

when PD = average population density within the entire urban area (4.213 person/acre for Atlanta).

From this curve, the least-cost for accomplishing any level of BOD control can be easily determined. Similar curves can be generated for percent runoff control. Note that for wet-weather quality control, significant diseconomies of scale exist because of the disproportionately large sized control units needed to capture the less frequent large runoff

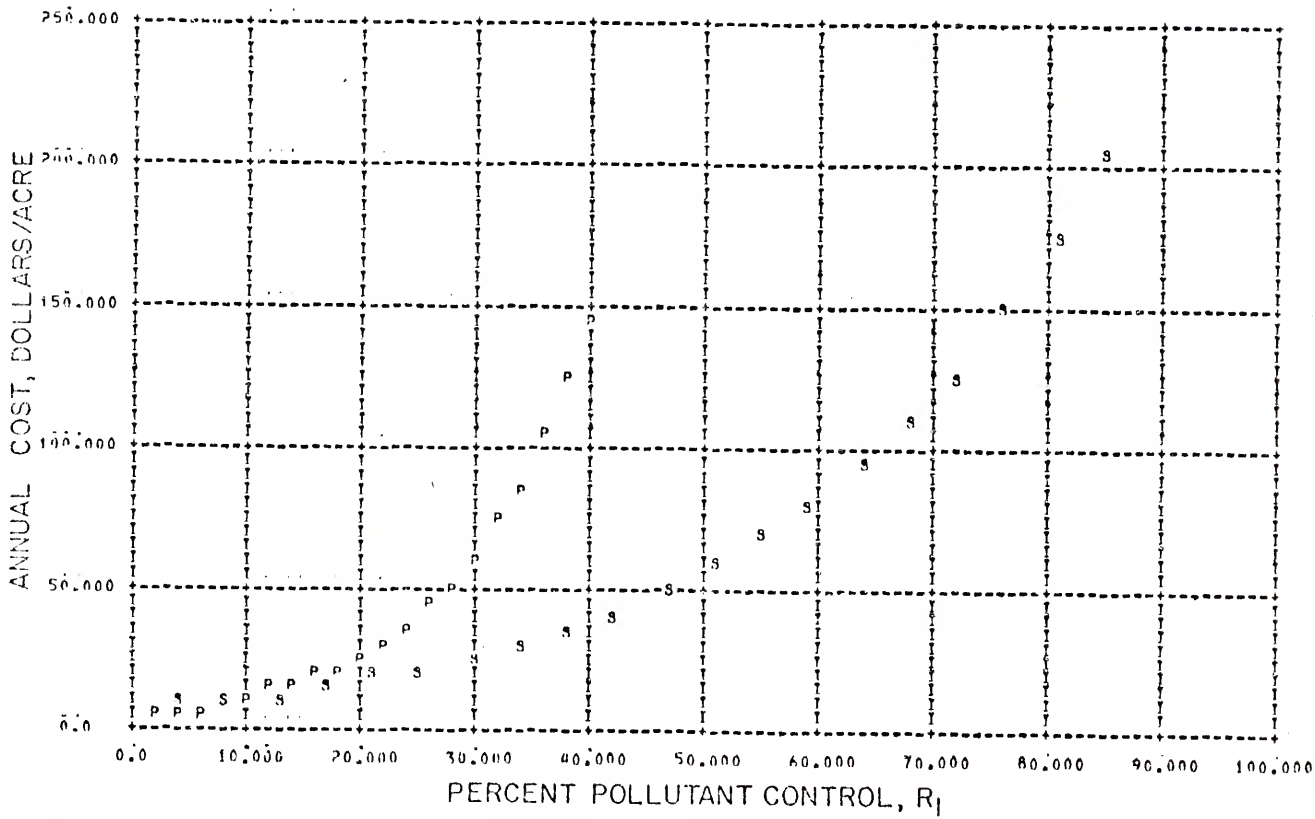


Figure 5-10. Control Costs for Primary and Secondary Devices as a Function of Percent BOD Removal—Region IV: Atlanta

volumes. The curves shown in Figure 5-10 can be approximated by the function of the form

$$Z = r_1 e^{R_1 r_2} \qquad (5.25)$$

where Z = total least annual cost for pollutant control R_1, dollars per acre;

r_1, r_2 = parameters;

R_1 = percent pollutant control, $0 \leq R_1 \leq \bar{R}_1$; and

$\bar{R}_1$ = maximum percent pollutant removed.

Fitting equation (5.25) to the curves presented in Figure 5-10 results in the following equations:

For primary device, $Z = 4.429e^{0.088R_1}$ (5.26)

For secondary device, $Z = 7.666e^{.039R_1}$. (5.27)

The correlation coefficient associated with equation (5.26) is 1.0 while for equation (5.27), the correlation coefficient is 0.999. Equations (5.26) and (5.27) yield the annual cost for percent BOD control R_1 in dollars per acre for Atlanta. It should be noted that the above equations are based on several assumptions, most important of which are (1) constant efficiency of treatment device irrespective of flow, (2) no treatment occurs in storage, (3) storage

and treatment costs are linear and as presented above; and (4) the first flush assumption built into the STORM model.

The analysis presented above can be illustrated by applying it to the hypothetical planning area. According to the control criteria listed in Table 2-9, city 6 is required to make provisions for 30 percent BOD control from wet-weather flows. The design data for city 6 are presented in Table 5-6. Since the hypothetical planning area is located in the southeastern part of the United States, it falls in region IV. The values of various parameters for the isoquant equations, using Atlanta as the test city, assuming first flush (Table 5-5), and applying equations (5.14) through (5.18) are as follows:

$$a = 0.0000252 \text{ in-hr}^{-1} (\%R)^{-1};$$

$$b = 0.004246 \text{ in-hr}^{-1};$$

$$h = 0.044997 (\%R)^{-1};$$

$$d = 133.8519 \text{ in}^{-1}; \text{ and}$$

$$f = 0.0323855 (\%R)^{-1}.$$

The following values of c_S and c_T are used.

$$c_S = 122e^{0.16PD} = 122e^{0.16(8.33)} = \$460/\text{acre-inch; and}$$

Table 5-6

Design Data for Wet-Weather Quality Control
Hypothetical Planning Area

	City 6
Total Area (acres)	15,000
Developed Area (acres)	13,000
Population	125,000
Average Population Density - total area (persons/ac)	8.33
Annual Precipitation (in)	44.0
Annual Runoff (in)	22.1
Annual BOD[a] - #/year-ac	100

[a]Computed from equation (2.3) and assuming approximately 50% of the area on combined sewers.

c_T = \$9800 per acre-in/hr for a secondary device.

Also for a secondary device,

$$\eta = 0.85,$$

therefore from equation (5.13)

$$R = \frac{R_1}{\eta} = \frac{30}{0.85} = 35.3 \text{ percent.}$$

Substituting the above value of R in equation (5.12) yields the following isoquant equation for city 6:

$$T = 35.3a + be^{h(35.3)} - (de^{-f(35.3)})S. \qquad (5.28)$$

Values of a, b, h, d and f have already been estimated. The above equation (5.28) can therefore be utilized to determine the treatment rate associated with the assumed or available storage volumes for accomplishing 30 percent BOD control. The optimal combination of storage volume and treatment rate can be computed from equations (5.22) and (5.23) to yield

$S^* = 0.069$ inches; and

$T^* = 0.002$ in/hr

Therefore, the annual cost of accomplishing 30% BOD control is \$51.00 per acre using the least-cost strategy. Further, the equation for optimal cost for various levels of wet-weather control is as follows:

$$Z = 15.8e^{.039R_1} \quad \text{using a secondary device.} \qquad (5.29)$$

5.6 Summary

This chapter has presented procedures for formulating strategies for wet-weather quantity and quality control and for determining the annual costs associated with these strategies. The analysis of wet-weather quantity and quality control problems may also involving piping-treatment/storage tradeoffs. The procedures presented in this chapter did not specifically address such problems. However, the analysis would be similar to that presented in Chapter 4. Therefore procedures presented in Chapter 4 in conjunction with those presented in this chapter could be utilized for conducting such an analysis.

The previous chapter as well as this chapter assumed that each purpose was to be accomplished independently. The next chapter presents procedures for formulation multipurpose wastewater management strategies.

CHAPTER 6

MULTIPURPOSE WASTEWATER MANAGEMENT

6.1 Problem Definition

The primary objective in multipurpose wastewater management is to utilize facilities designed for a single purpose towards accomplishing other purposes. When such a possibility exists, either the annual costs associated with accomplishing other purpose(s) can be reduced or a higher level of control can be achieved for the fixed annual costs thereby yielding a more cost-effective strategy. The control of domestic wastewater has received priority in the past and is again emphasized in the guidelines for area-wide waste treatment and management. Thus, treatment facilities must be provided for dry-weather quality control. A similar situation usually exists for wet-weather quantity control as flooding problems require regulation of the wet-weather quantity usually in the form of on-site or off-site storage. The discussion presented in the previous chapter shows that optimal wet-weather quality control requires a combination of storage and/or treatment. Therefore, if the dry-weather treatment facilities are

capable of handling a portion of wet-weather flows, the volume and pollutant loadings to be handled by the wet-weather quality control devices can be reduced by this amount. This, in turn, reduces the storage and/or treatment required for wet-weather quality control. Similarly, if one utilizes the storage required for wet-weather quantity control to accomplish the wet-weather quality control as well, the storage requirements for wet-weather quality control are reduced. In this instance, annual costs for dry-weather quality control, and wet-weather quantity control remain the same as for single purposes. However, the annual costs associated with wet-weather quality control are reduced from what these would be if this purpose was accomplished separately.

Other incentives for multipurpose wastewater management exist. Among these are the possibility of utilizing wet-weather quality control devices, during dry-weather, for providing a higher level of domestic wastewater treatment or a reduction in the level of treatment required for dry-weather quality control by removing an equivalent amount of pollutants from wet-weather flows.

In this chapter, various strategies for multipurpose wastewater management are presented. Optimization procedures for evaluating the annual costs associated with various strategies are outlined. The application of these procedures is illustrated by the hypothetical planning area example.

6.2 Strategies for Multipurpose Wastewater Management

Several strategies for accomplishing the three purposes of urban water management can be formulated. One such strategy is to accomplish each purpose independently. The annual costs associated with this strategy can be evaluated in accordance with the procedures discussed in Chapters 4 and 5. Another strategy is to accomplish all three purposes in a joint manner by utilizing all the facilities to their maximum. Other alternative strategies are comprised of various combinations of single purpose and multipurpose strategies whereby one purpose is accomplished independently while the remaining two purposes are accomplished jointly. Another strategy that should be investigated, especially where waste load allocation requires an advanced level of treatment for dry-weather treatment, is the feasibility of a lower level of dry-weather treatment in conjunction with wet-weather quality control.

Traditionally, dry-weather treatment facilities are designed to handle only domestic wastewater with little or no provision for treating stormwater. During the last few years, under the sponsorship of the Environmental Protection Agency, intensive studies have been conducted to develop treatment devices capable of operating efficiently under high flow conditions. As a result, several demonstration facilities have been constructed for the sole purpose

of treating combined sewer overflows. Many of the unit processes, successfully demonstrated, are similar to those commonly utilized in dry-weather treatment. From the results of these demonstration facilities, it appears technically feasible to design dry-weather treatment units to handle larger flow rates during wet weather. Research was therefore undertaken to investigate this possibility. The following discussion reviews various dry-weather treatment units and illustrates how these units could be modified to handle wet-weather flows in light of the data available from demonstration facilities.

Preliminary Treatment

Preliminary treatment of dry-weather flows involves removal of coarse solids and grit from wastewater in order to protect the wastewater treatment equipment. The treatment is provided by installing bar racks and grit tanks in series.

In most dry-weather installations, mechanically cleaned bar racks are utilized with provisions for bypassing in case of emergency. For this purpose, one or more bypass channels with manual screens are installed to facilitate operation when the mechanical screen is inoperative. Both the mechanical screen as well as manual screens are designed to handle peak dry-weather flow, of the order of two to five times the average dry-weather flow. Bar screens are

also required for treating wet-weather flows. If both the mechanical screen and the manual screen at the dry-weather facility are utilized, the same installation can approximately handle twice the design peak flow with one bypass channel and three times the design peak with two bypass channels. Manual screens are relatively inexpensive.

The grit tank is a device for controlling the velocity of flow in order for the heavy particles in wastewater to settle out. It consists of a long narrow channel equipped with grit collectors and velocity control. In recent years, aerated grit chambers have become popular. These units are designed for a detention time of three to five minutes at peak flow. Grit removal is also required for wet-weather flows. Therefore, if the dry-weather grit chamber is designed for a higher peak flow, the same facilities can be utilized for grit removal during dry- as well as wet-weather periods.

Primary Treatment

For domestic wastewater, primary treatment follows preliminary treatment and is comprised of a sedimentation tank equipped with bottom sludge scrapers and top skimmers. During the course of treatment, lasting about two hours at average design flow, heavier solids settle to the bottom while the lighter materials float to the top. Primary treatment reduces the BOD_5 and SS concentration of the

wastewater. Generalized removal curves for BOD_5 and SS versus detention times are shown in Figure 6-1 (Fair, Geyer and Okun, 1968). These removals can be approximated by equation (2.4). The sedimentation tank is usually designed for two hours' detention at average design flow yielding approximately 60 percent SS removal and 35 percent BOD_5 removal.

Primary treatment has also been utilized for treating wet-weather flows. Installations of this nature presently exist at Jamaica Bay in New York, Humboldt Avenue in Milwaukee, Cottage Farm in Boston, and Whittier Narrow in Columbus, Ohio (Lager and Smith, 1974). The detention time for treatment varies from 10 minutes at Cottage Farm to 24 minutes at Whittier Narrow. The reported removal efficiency range is 15-45 percent for SS and 15-35 percent for BOD_5. Since a sedimentation tank designed on the basis of 30 minutes' detention will process four times more flow and pollutants than the tank designed on a two-hour detention, the mass of pollutants removed would be higher with the half-hour detention facility. Thus, if the sedimentation tank of the dry-weather treatment facility is designed to handle larger peak flows than conventionally used, the tank can be utilized for treatment during wet-weather periods.

A primary treatment process that is not too common in dry-weather treatment but has been used for treating industrial wastewaters as well as wet-weather flows is

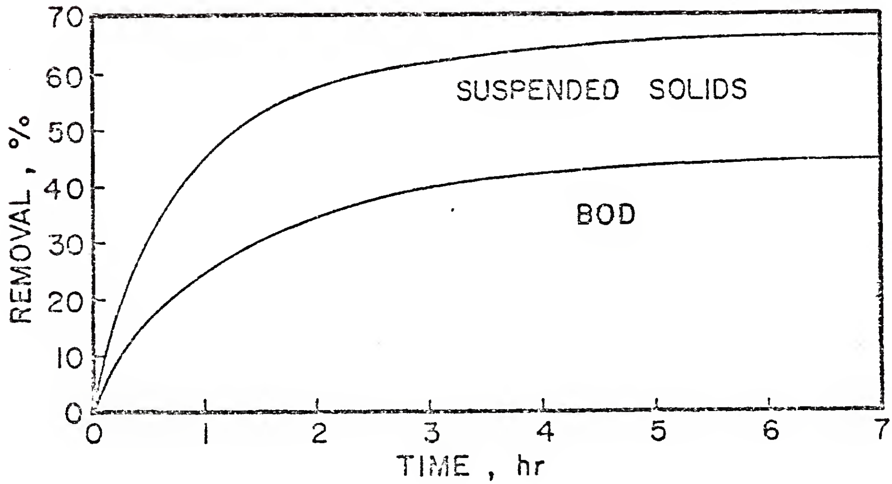


Figure 6-1. Pollutant Removal in a Primary Tank as a Function of Detention Time (Fair, Geyer and Okun, 1968)

dissolved air flotation. In this process, the waste flow or a portion thereof is pressurized in the presence of sufficient air to approach saturation. When this pressurized air-liquid mixture is released to atmospheric pressure in a flotation tank, minute air bubbles are released from the solution and suspended solids are floated to the surface where they are skimmed. This process has been used for wet-weather treatment at Milwaukee and Racine, Wisconsin, and San Francisco (Lager and Smith, 1974). The principal parameters that affect the removal efficiencies of the process are (1) detention time, (2) amount of air dissolved in the flows, and (3) chemical addition. Detention time is the most critical parameter affecting SS and BOD_5 removals of the dissolved air flotation process. The process with 15 to 20 minutes' detention yields BOD_5 and SS removal efficiencies comparable with a sedimentation tank with two hours' detention.

The flotation tank is similar to the sedimentation tank in structure and equipment. Thus, if the pressurization equipment required for the dissolved air flotation process is provided, the sedimentation tank designed for dry-weather treatment can also be utilized as a flotation tank during wet weather. This feature allows the handling of six to eight times the flow during wet weather without any reduction in removal efficiencies.

Secondary Treatment

Secondary treatment of municipal wastewater is usually accomplished by either a high rate trickling filter process or an activated sludge process.

In the high rate trickling filter process, the primary treated effluent is sprayed over rock media at a rate of about 30 million gallons per acre per day. The organic removal is accomplished by the microorganic growth attached to the filters. The effluent from the filter is settled in a clarifier. Some sort of recirculation is practiced in all high rate filters.

In the conventional activated sludge process, the raw waste or primary treated effluent is aerated for a period of about six hours in an aeration tank in the presence of activated sludge. The activated sludge is the solids that settle in the secondary clarifier. A portion of the solids are recirculated to the aeration tank while the balance is conveyed to the digester. The contents of the aeration tank, called the mixed liquor, are allowed to settle in the secondary clarifier. The secondary clarifier is usually designed for a detention time of two hours and an overflow rate of up to 1,000 gallons per day per square foot. The clarifier effluent is disinfected prior to discharge to the receiving waters.

Biosorption, more commonly known as contact stabilization, is a slight modification of the activated sludge

process. In this process, the raw waste or the primary treated effluent is aerated in a contact tank for a period varying from 30 minutes to three hours. Then the effluent is sent to the secondary clarifier. The settled activated sludge is then aerated in a stabilization tank for a period of four to six hours prior to its recirculation into the contact tank. When this sludge is brought in contact with the wastewater in the contact tank, removal of the pollutants occurs by adsorption. An advantage of this process is the reduction of the size of tankage required for aeration and as a consequence the process is less expensive than the conventional activated sludge process. Because of this advantage, the biosorption process is being used more and more for providing secondary treatment to domestic wastewater. Many of the existing conventional activated sludge facilities are being converted to the contact stabilization process in order to increase their design capacities.

Up until a few years ago, biological treatment of wet-weather flows was not considered feasible primarily because of the large volumes involved and the sensitivity of the process to shock loads. Recently, however, both the trickling filter as well as the contact stabilization process have been successfully applied for treating wet-weather flows. The trickling filter installation is located in the Borough of New Providence, New Jersey (Homack et al.,

1973), while the contact stabilization process is being utilized at Kenosha, Wisconsin (Agnew et al., 1975).

The New Providence trickling filter installation has a rock media filter and a plastic media filter. During dry weather, the filters operate in series while during wet weather the filters operate in parallel. The hydraulic loadings on the plastic media filter, during wet weather, range from 80 to 150 million gallons per acre per day. The rock media filter was subjected to hydraulic loadings ranging from 15 to 53 million gallons per acre per day. Removal efficiencies of the rock media at 15 million gallons per day per acre were comparable to those obtained in the plastic media with hydraulic loadings of 80 million gallons per acre per day. Thus, plastic media filters can handle substantially higher flows than the rock media filters. Therefore, use of plastic media filters in dry-weather installations can enhance their capability to handle large flows during wet weather.

The Kenosha facility was built in parallel with an existing 23 mgd conventional activated sludge dry-weather plant and has a design flow of 20 mgd. The combined sewer overflow in excess of the design capacity of the 23 mgd (design flow of dry-weather treatment facility) is diverted to this treatment facility. The overflow treatment facility is comprised of a grit removal tank, contact tank, stabilization tank and the secondary clarifier. Since the facility

is designed to treat combined sewer overflows, the operation of the facility is intermittent. Consequently, a reservoir of waste activated sludge obtained from the dry-weather plant is maintained in a biologically active state to initiate biological treatment in the overflow facility when required due to a storm event.

The results of the Kenosha installation indicate that separate treatment of wet-weather flows is feasible provided that adequate quantities of dry-weather-activated sludge are available during wet weather. The volume of wet-weather flow that can be treated is dependent on the amount of dry-weather-activated sludge that is available.

The contact stabilization process for treating dry-weather sewage flow is normally designed to provide a contact time ranging from 30 minutes to three hours. Thus, if such a facility is designed on the basis of a two-hour contact time during dry weather, 10 to 20 minutes' contact time required during wet weather would enable the same tank to process six to 12 times the dry-weather flow. The efficiency of the process is also dependent on the organic loading. The effect of smaller contact time and higher organic loading during wet weather is to reduce the efficiency of treatment somewhat. At Kenosha, BOD removal efficiency of 83 percent was obtained as against 90 percent normally achieved for dry-weather treatment by contact stabilization.

Since the quantities of wet-weather flow that can be treated depend upon the quantities of dry activated sludge that can be stored in a viable state, the maximum flow treated by the contact stabilization process will be limited either by the minimum detention time or the maximum amount of dry-weather-activated sludge production per day. At Kenosha, it was found that activated sludge from this dry-weather plant can be stored for a period of five days without any loss of viability. The quantities of stored sludge must be sufficient to maintain a mixed liquor concentration (MLSS) of 2500 ppm. Based on average sludge production data for dry-weather plants, the estimated maximum flow that can be treated during wet weather is of the order of eight to ten times dry-weather flow.

The effluent from either trickling filters or the activated sludge process is usually settled in a secondary clarifier. This unit is similar in many respects to a primary sedimentation tank. However, its operation is more critical than a primary clarifier. If a secondary clarifier does not perform efficiently, greater amounts of SS and BOD_5 may be carried in the discharge. Thus, the unit cannot be converted into a high rate operation and additional clarification capacity must be added if the facility is to handle large quantities of wet-weather flows. As an alternate, the use of the dissolved air flotation process to increase the capability of the secondary clarifier

appears feasible. Studies of the feasibility of using the dissolved air flotation process to clarify activated sludge tank effluent (Mulbarger and Huffman, 1970) indicate that it is not successful. It has been postulated that the use of dissolved air flotation shears the highly flocculated aeration tank effluent resulting in poor efficiencies of the dissolved air process. However, in the contact stabilization process, the effluent from the contact tank has not had an opportunity to flocculate and therefore the dissolved air flotation appears feasible.

In summary, utilization of the above concepts can allow greater volumes of wet-weather flows to be treated at the domestic wastewater treatment facilities than would be possible if only the normal excess capacity is utilized because without utilizing these concepts the facility would operate as a conventional device during dry as well as wet weather. If these concepts are incorporated in the domestic wastewater facilities at the time of their design, very little additional cost would be involved. Based on the plant design capacity and the actual dry-weather flow, the treatment capacity available for wet-weather flow can be determined.

6.3 Optimal Sequence of Dry-Weather and Wet-Weather Quality Control

As stated previously, control of domestic wastewater has, in the past, received priority over wet-weather quality

control. As a result, many of the urbanized areas have been required to provide advanced treatment of their domestic wastewater in order to further reduce the pollutants discharged from this source. In those urban areas where the water quality is affected by both dry- and wet-weather flows, it may be more cost effective to institute controls on wet-weather quality prior to requiring increased level of domestic wastewater treatment. Therefore, it is necessary to develop procedures for determining the optimal sequence of dry-weather and wet-weather quality control. Let

TC^{ψ} = annual cost of dry-weather treatment for treatment level ψ, dollars per year;

IC_D = incremental cost of increasing the dry-weather treatment level from ψ to ϕ, dollars per year;

M_D = dry-weather BOD, pounds per year;

η^{ψ} = efficiency of dry-weather treatment for treatment level ψ $(0 \leq \eta \leq 1)$;

η^{ϕ} = efficiency of dry-weather treatment for treatment level ψ $(0 \leq \eta \leq 1)$; and

IM_D = incremental dry-weather BOD removal when the level of treatment is increased from ψ to ϕ, pounds/year.

Then

$$IC_D = TC^{\phi} - TC^{\psi} \tag{6.1}$$

$$IM_D = M_D(\eta^{\phi} - \eta^{\psi}) \tag{6.2}$$

and

$$c_D = \frac{IC_D}{IM_D} \tag{6.3}$$

where c_D = incremental or marginal cost of increasing domestic wastewater treatment from level ψ to ϕ, dollars/pound BOD removal.

The cost of wet-weather quantity control as given by equation (5.29) can be written as follows:

$$Z = r_1 e^{r_2 R_1} \tag{6.4}$$

where Z = annual cost of wet-weather quality control, dollars/year;

R_1 = percent of wet-weather BOD control;

r_1 = constant; and

r_2 = constant.

Let M = annual wet-weather BOD generated, pounds/acre;

M_1 = pounds of wet-weather BOD removed.

Then

$$R_1 = \frac{M_1}{M} \times 100 \tag{6.5}$$

Note that M can be estimated using equations presented in Chapter 2.

From equations (6.4) and (6.5)

$$Z = r_1 e^{r_2 \left(\frac{100(M_1)}{M}\right)} . \tag{6.6}$$

Therefore the marginal cost of wet-weather BOD removal for this convex function is

$$\frac{dZ}{dM_1} = r_1 r_2 \frac{100}{M} e^{r_2 \left(\frac{100(M_1)}{M}\right)} \tag{6.7}$$

Equating the marginal costs of dry-weather treatment and wet-weather treatment yields

$$r_1 r_2 \frac{100}{M} e^{r_2\left(\frac{100(M_1)}{M}\right)} = c_D$$

or

$$M_1{}^* = \frac{M}{100r_2} \ln \frac{c_D M}{100r_1 r_2} \qquad (6.8)$$

Thus, one can determine the pounds of wet-weather BOD, $M_1{}^*$, that should be removed prior to increasing the degree of wastewater treatments from level ψ to level ϕ. This procedure allows the optimal sequence of dry- and wet-weather quality control to be determined.

The procedure can be illústrated by application to city 6 of the hypothetical planning area. If the existing 10 mgd wastewater treatment facility was to be upgraded to provide tertiary treatment, the incremental cost of tertiary treatment, using equation (6.1), is $511,000 per year. From Table 2-9, the design BOD of the plant is 20,000 pounds per day and the efficiency of treatment is 90 percent. Assuming that the efficiency of tertiary treatment is 95 percent, the additional BOD removal due to tertiary treatment is 1000 pounds per day or 365,000 pounds per year. Therefore, the marginal cost of tertiary treatment would be $1.40 per pound of BOD removed, and the value of M_1^* can be computed as follows:

M = 100 #/year-ac (Table 5-6)

c_D = $1.40/pound of BOD.

From equation (5.29), the optimal cost for wet-weather quality control for city 6 is given by the following equation

$$Z = 15.8e^{0.039R_1}$$

Using equation (6.8) yields:

$$M_1^* = 21.0 \text{ pounds/year-ac.}$$

Therefore,

$$\frac{M_1^*}{M} = 0.21$$

Thus, approximately 21 percent of the wet-weather BOD control should be instituted prior to requiring tertiary treatment of domestic wastewater.

6.4. Optimal Multipurpose Wastewater Management

The costs of urban wastewater management can be reduced by integrating various purposes. Optimization procedures for evaluating annual costs of these strategies are discussed below.

Let Z = cost of dry-weather control at a secondary plant, dollars per year

= amortized capital costs + annual O and M costs.

From cost data presented in Table 2-5,

$$Z = 118{,}000\ (\hat{D})^{0.77} + 55{,}000\ D^{0.83} \qquad (6.9)$$

where $\hat{D}$ = design capacity of the plant, mgd; and

D = actual sewage flow, mgd.

Let E = excess capacity of dry-weather plant, in/hr; and

Z_1 = annual cost of dry-weather quality control, dollars per acre.

If per capita wastewater flow is known, $\hat{D}$ and D can be expressed in terms of population density, PD. Therefore,

$$Z_1 = J(PD) \tag{6.10}$$

where

Z_1 = annual cost of dry-weather quality control, dollars per acre;

PD = population density, persons per acre; and

J = constant.

Also,

Z_2 = annual cost of wet-weather quality control, dollars per acre, assuming linear costs

$$= c_S S^* + c_T(T_1 + (T_2 - T_1)e^{-KS^*}] \tag{6.11}$$

where all terms are as defined earlier, and

Z_3 = annual cost of wet-weather quantity control, dollars per year,

$$= c_S V \tag{6.12}$$

where V = storage volume required for wet-weather quantity control, inches

$$= (B_D - B_u)I$$

B_D = runoff coefficient in developed state;

B_u = runoff coefficient in undeveloped state; and

I = 24-hour rainfall for design frequency, inches.

If dry-weather quality control and wet-weather quality control are integrated, then

Z_{12} = joint annual cost of two purposes, dollars per acre,

$$= J(PD) + c_E E + c_S S^* + c_T[(T_1 - E) + (T_2 - T_1)e^{-KS^*}] \qquad (6.13)$$

where E = excess capacity available at dry-weather plant, inches per hour; and

c_E = cost of treating E at dry-weather plant.

If wet-weather quality control is integrated with wet-weather quantity control, then

Z_{23} = joint annual cost of two purposes, dollars per acre,

$$= c_S S_2^* + c_T[T_1 + (T_2 - T_1)e^{-KS_2^*}] \qquad (6.14)$$

where

$$S_2^* = \max \begin{cases} V \\ \frac{1}{K} \ln\left[\frac{c_T}{c_S} K(T_2 - T_1)\right] \\ 0 \end{cases} \qquad (6.15)$$

It is assumed that dry-weather control cannot be integrated with wet-weather quantity control. Therefore,

Z_{13} = joint annual cost of two purposes, dollars per acre,

$$= Z_1 + Z_3 \tag{6.16}$$

$$= J(PD) + c_S V \tag{6.17}$$

If all three purposes are integrated, then

Z_{123} = joint annual cost of all three purposes

$$Z_{123} = J(PD) + c_E E + c_S S_2^* + c_T[(T_1 - E) + (T_2 - T_1)e^{-KS_2^*}] \tag{6.18}$$

Thus, the procedure outlined above can be utilized to evaluate the costs associated with various multipurpose strategies.

The above optimization procedure can be illustrated by applying it to city 6 of the hypothetical planning area. Assume that the city decides to increase the treatment capacity of its existing sewage treatment facility to 15 mgd. The anticipated domestic wastewater flow to the facility is 12.5 mgd. Thus

$\hat{D}$ = 15 mgd

D = 12.5 mgd

E = 2.5 mgd = 0.000295 inches per hour.

Let $V = 0.25$ inches;

$c_E = \$2610$ per inch per hour; and

$R_1 = 30$ percent.

Then, from equation (6.9)

$$Z_1 = 118{,}000(15)^{.77} + 55{,}000(12.5)^{.83}$$
$$= \$1{,}406{,}000 \text{ or } \$108 \text{ per acre.}$$

From the example presented in Chapter 5,

$Z_2 = \$51$ per acre; and

$c_S = \$460$ per acre-inch

Using equation (6.12),

$$Z_3 = c_S V$$
$$= (460)(0.25) = \$115 \text{ per acre.}$$

By using the isoquant parameters established for city 6 in the example presented in Chapter 5 and applying equations (6.13), (6.14), (6.15), (6.16), and (6.17),

$Z_{12} = \$157$ per acre;

$Z_{23} = \$124$ per acre;

$Z_{13} = \$223$ per acre; and

$Z_{123} = \$230$ per acre.

The annual cost of various multipurpose strategies is presented in Table 6-1. It can be seen that, in this case, multipurpose planning reduces the cost of wastewater management from that which would be incurred if each purpose was to be accomplished separately.

6.5 Summary

This chapter has presented concepts which can be utilized to increase the capability of existing domestic wastewater treatment facilities for handling wet-weather flows. A procedure for determining the level of wet-weather quality control which yields the same marginal cost as the incremental cost between secondary and tertiary treatment was also outlined. Optimization procedures for evaluating various multipurpose wastewater strategies were also outlined. The procedures presented in this chapter are quite useful in examining cost-effective strategies for dry-weather and wet-weather control. The next chapter outlines the efficiency and equity considerations that are important in urban wastewater management problems involving several purposes or groups.

Table 6-1

Annual Costs for Various Multipurpose Wastewater Management Strategies: City 6--Hypothetical Planning Area

Strategy Number	Description	Annual Cost-$/acre							
		Z_1	Z_2	Z_3	Z_{12}	Z_{13}	Z_{23}	Z_{123}	Total
1	Each purpose is accomplished independently	108	51	115					274
2	Purposes 1 & 2 accomplished jointly with purpose 3 accomplished independently			115	157				272
3	Purposes 1 & 3 accomplished jointly with purpose 2 accomplished independently		51			223			274
4	Purpose 1 accomplished independently with purposes 2 & 3 accomplished jointly	108					124		232
5	All purposes accomplished jointly							230	230

CHAPTER 7

INTEGRATED EFFICIENCY/EQUITY ANALYSIS

7.1 Problem Definition

Chapters 4, 5 and 6 presented procedures for formulating alternative strategies for accomplishing various purposes of urban wastewater management and for determining their associated resource (annual) costs. These procedures assumed that each of the alternative strategies, if implemented, would satisfy all regulatory requirements, i.e., control criteria. As stated in Chapter 3, each of these strategies may have different environmental and social impacts. A strategy with least annual costs may have more adverse impacts than another strategy (for accomplishing the same purpose) with higher annual costs. Since the stated objective of urban wastewater management is the selection of a cost-effective solution, it becomes necessary to evaluate these impacts for various strategies. However, many of these impacts can only be quantified in nonmonetary terms. Based on goals and objectives of an area under consideration, it may be possible to select a single parameter to express these

objectives (Dinius, 1972; Council on Environmental Quality, 1972). Environmental and social impacts of various strategies can then be expressed in terms of this single parameter. Then the least cost solution can serve as a base against which increased costs and impacts of the other alternatives can be compared for the purpose of selecting the best plan for the area. This analysis can be used to display the noninferior set of solutions for this multiobjective problem. The selection of the cost-effective solution then depends on the decision maker's preferences and the political and implementation feasibility of a plan from this noninferior set of solutions.

Implementation feasibility is dependent in part on how the costs of a plan are shared among various groups (called cost-sharing) and among various purposes (called cost-allocation). If a group or a purpose feels that the costs assigned to it in a plan are not fair, it is difficult to implement such a plan. Some of the equity questions that arise in environmental quality management were discussed in Section 3.4 A mathematical formulation of the equity problem presented in Section 3.5 showed it to be also a multiobjective problem. In this case, however, the objectives are in monetary terms and the noninferior set can be generated by economic analysis or by using multiobjective mathematical programming techiques. Another equity question that may arise is the allocation

of capacities among different purposes or groups. As shown in Section 3.2, the efficient solution for accomplishing two purposes is by joint utilization of the facilities. However, unless the allocation is fair, the solution may not be implementable. In this case also, economic analysis results in a noninferior set of solutions. In order to determine the optimal allocation or the best compromise solution, the economic analysis relies on product prices or marginal benefits while multiobjective analysis utilizes the indifference curve approach. Since this information is not available in urban wastewater management, other procedures for resolving the distributional aspects of such problems have been proposed or utilized. As will be illustrated, these methods do not necessarily result in an equitable arrangement.

The purpose of this chapter is to first present efficiency/equity criteria which encourage the implementation of the selected plan. A review of multiobjective solution techniques as well as conventional cost-sharing/cost-allocation procedures is undertaken and their deficiencies noted. A brief discussion of cooperative N-person game theory is presented and it is shown that application of the concepts derived from game theory can result in a distribution which has desirable equity properties. These concepts are then related to conventional procedures.

7.2 Efficiency/Equity Criteria

As stated above, the objective of such criteria is to encourage the implementation of an efficient or cost-effective solution as the case may be. These criteria will be developed by means of examples. First consider the case of allocation of capacities between two purposes as discussed in Section 3.1. The contract curve OABO shown in Figure 3-6 results in the transformation curve or noninferior set PP'Q'Q as shown in Figure 7-1. Point A represents the status quo if each purpose is accomplished independently. Suppose that the economic analysis was to be used to determine the optimal allocation. Assume that the marginal benefits are such that the solution falls in the range PP'. This implies that a reduction be effected in dry-weather BOD removal from its current status. Such a solution may be difficult to implement for several reasons, e.g. (1) it may not meet regulatory requirements; or (2) it disregards existing ownership. Similar reasoning could be applied to any solution falling in the range Q'Q. It is, therefore, seen that the existence of a status quo point delimits the noninferior set of solutions. In the present case, the solution must lie on curve P'Q'. Any solutions on the right of this curve are infeasible, while those on the left are inefficient. The solutions to the left of AP' and below

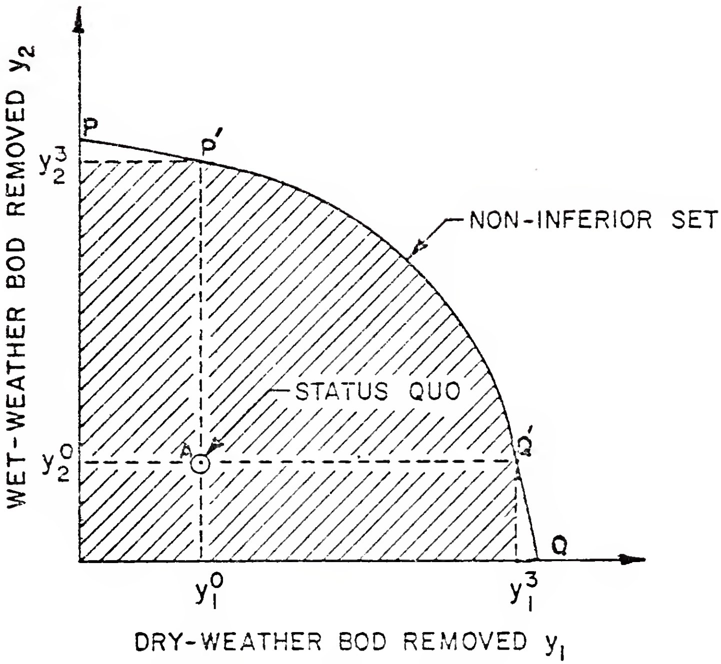


Figure 7-1. Illustration of Equity Criteria for Allocation of Capacity Example

AQ' may be viewed as inequitable. The above resource allocation problem can be mathematically stated as follows:

$$\begin{aligned} &\text{maximize } (y_1, y_2) \\ &\text{subject to} \\ &\qquad g(y_1, y_2) = 0 \\ &\qquad\qquad y_1 \geq y_1^o \\ &\qquad\qquad y_2 \geq y_2^o \\ &\qquad\quad y_1, y_2 \geq 0. \end{aligned} \tag{7.1}$$

In problems involving apportionment of project costs among the users, it is necessary that, unless a subsidy is provided, full costs be obtained from the users before a project can be implemented. For example, consider a case involving two cities faced with constructing a wastewater treatment plant. In Figure 7-2, the total project costs, C_{12}, are represented by line PQ. In order for the total costs to be covered, the solution must lie on line PQ. Line PQ may also be viewed as the noninferior set as any move along this line increases the costs to one city while decreasing it for the other city. If the cost-sharing point is to the right of this line, the revenues will exceed the project costs while on the left of the line, insufficient revenues are generated. The problem is, how should the costs be apportioned?

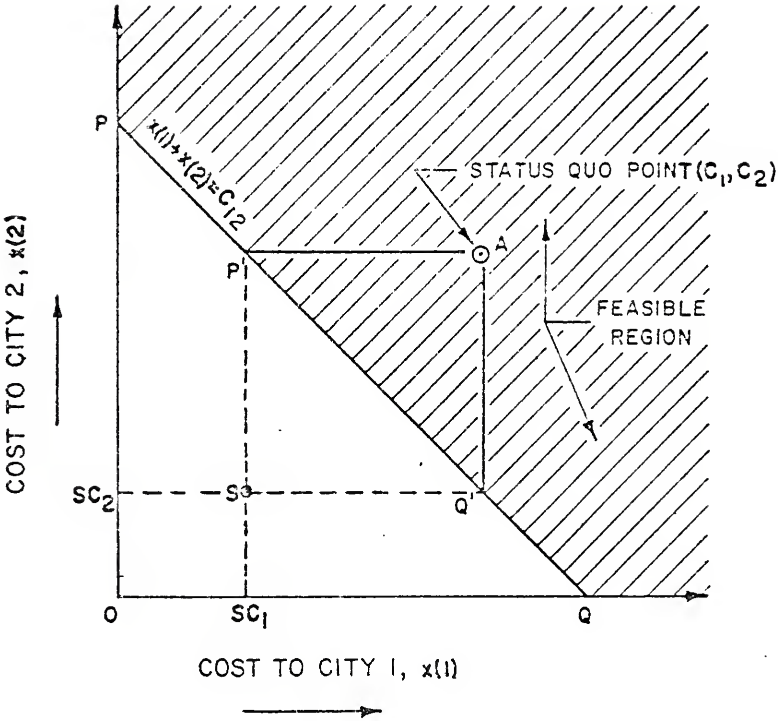


Figure 7-2. Illustration of Equity Criteria for Cost-Sharing/Cost-Allocation Example

Several authors (Hazelwood, 1951; Brandis, 1953; Loehman and Whinston, 1971; James and Lee, 1971) have advocated the use of marginal or incremental cost pricing as a necessary condition for efficiency and fairness. These costs are defined as follows:

$$SC_i = C(N) - C(N - \{i\}) \tag{7.2}$$

where SC_i = separable or marginal cost to i;

$C(N)$ = total project cost with n users; and

$C(N - \{i\})$ = total project cost with user i excluded.

In Figure 7-2, let point A denote the costs given by $C(N - \{i\})$. For the case involving two users, this point corresponds with the costs of serving each user independently. Given point A, point S is uniquely determined as shown in Figure 7-2. This point denotes the separable or marginal costs. Since this point lies on the left of the line PQ, it is evident that the marginal cost pricing will not cover the total project cost. The portion of the project cost that is not covered is given by

$$NSC = C(N) - \sum_i SC_i \tag{7.3}$$

These costs are usually called joint or nonseparable costs. A negative NSC implies that point A lies on the left of the line PQ and, therefore, the joint arrangement is economically inefficient. Thus, the only case of interest is when $NSC \geq 0$.

From Figure 7-2, it is also evident that the cost sharing point must lie on the line segment P'Q'. If the cost sharing point lies on the line PQ but outside the line segment P'Q', then the total project cost is covered. However, one of the users will be assigned a cost greater than his cost of going alone. If point A is denoted by status quo point, it is seen that the existence of the status quo point results in delimiting the set of solutions from line PQ to the line segment P'Q'. This problem can be mathematically stated as follows:

$$\begin{aligned} &\text{minimize } (x(1),\ x(2)) \\ &\text{subject to} \\ &\quad x(1) + x(2) = C_{12} \\ &\qquad x(i) \leq C_i \qquad \forall i \\ &\qquad x(i) \geq SC_i \qquad \forall i \\ &\qquad x(i) \geq 0 \qquad \forall i \end{aligned} \tag{7.4}$$

where $x(i)$ = cost apportioned to i;

C_i = cost of serving i alone; and

C_{12} = total project cost.

Thus, from efficiency and equity standpoints, the solution in both cases must lie on the line segment P'Q' which is

the noninferior set. In the case of cost sharing, any solution ϕ_i on the line segment P'Q' can be represented by

$$\phi_i = SC_i + \beta_i(NSC) \qquad (7.5)$$

where $\phi_i \in x(i)$; and

β_i = fraction of the nonseparable cost assigned to i.

Note that

$$\sum_i \phi_i = C(N)$$

for $\sum_i \beta_i = 1.$

For two users,

$$SC_1 = C_{12} - C_2 \qquad (7.6)$$

$$SC_2 = C_{12} - C_1 \qquad (7.7)$$

and $NSC = [C_{12} - (C_{12} - C_2 + C_{12} - C_1)]$ (7.8)

Therefore, using equation (7.5),

$$\phi_1 = C_{12} - C_2 + \beta_1[C_{12} - (C_{12} - C_2 + C_{12} - C_1)]$$

$$= C_{12}(1 - \beta_1) + C_2(-1 + \beta_1) + \beta_1 C_1 \qquad (7.9)$$

and $\phi_2 = C_{12}(1 - \beta_2) + C_1(-1 + \beta_2) + \beta_2 C_2$ (7.10)

Also $\phi_1 + \phi_2 = C_{12}$

for $\beta_1 + \beta_2 = 1$

Applying the above approach to three users,

$$\phi_i = C(N)(1 - 2\beta_i) + C(N - \{i\})(-1 + \beta_i) + \beta_i \sum_{i\varepsilon \underline{S}} C(\underline{S}) \qquad (7.11)$$

where $C(N)$ = total project cost with three users;

$C(N - \{i\})$ = total project cost with user i excluded; and

$C(\underline{S})$ = total project cost for subcoalitions of 2 containing i.

Extending the above results to n participants yields,

$$\phi_i = C(N)[1 - (n-1)\beta_i] + C(N - \{i\})(-1 + \beta_i) + \beta_i \sum_{i\varepsilon \underline{S}} C(\underline{S}) \qquad (7.12)$$

where $\underline{S}$ = set of coalitions of order n-1 containing i.

Again

$$\sum_{i=1}^{n} \phi_i = C(N)$$

for $$\sum_{i=1}^{n} \beta_i = 1$$

Thus, given the noninferior set P'Q', apportionment of costs involves selection of appropriate values of β_i.

The manner in which this is accomplished in conventional cost-sharing/cost-allocation techniques is discussed in Section 7.4. The next section discusses how a unique solution,or the best compromise solution to multiobjective problems of the type presented above, is obtained by multiobjective soltuion techniques.

7.3 Multiobjective Solution Techniques

Multiobjective solution techniques designed to determine the best compromise solution to the multiobjective problems are categorized as interactive techniques. These techniques proceed from one noninferior solution to another, at the direction of the decision maker, until the best compromise solution is found or another condition for termination of the algorithm is met (Cohon, 1973). One of these techniques is the step method or STEM. The STEM procedure begins with n alternatives, each of which maximizes one of the n objectives. Thus, plan $\overline{X}_j$ is the solution to

$$\text{maximize } F_j(\overline{X}) \qquad (7.13)$$

$$\text{subject to}$$

$$g_i(\overline{X}) = 0 \quad \forall\, i$$

The maximum value of objective j, $F_j(\overline{X}_j)$ is denoted by M_j and values of other objectives when $\overline{X} = \overline{X}_j$ are given by $F_K(\overline{X}) = Z_{Kj}$ $(K \neq j)$. The results are specified in a payoff

matrix with M_j forming the diagonal of the matrix. M_j is defined as the "ideal" solution for objective j.

The STEM procedure then proceeds to determine the solution which is "nearest" in the minimax sense to the "ideal" solution, i.e., the maximum weighted difference, E, between objectives $F_j(\overline{X})$ and their respective maximum value, M_j; or

$$\begin{aligned} &\text{minimize } E \\ &\text{subject to} \\ &\quad E \geq W_j[M_j - F_j(\overline{X})] \qquad \forall j \\ &\quad g_i(\overline{X}) = 0 \qquad \forall i \end{aligned} \tag{7.14}$$

The weights W_j indicate the relative magnitude or significance of the deviations from the optimum. The solution to the first iteration is a plan, $\overline{X}_0$, which accomplishes a vector of objectives, $\overline{Z}_0 = [F_1(\overline{X}_0), F_2(\overline{X}_0),\ldots,F_n(\overline{X}_0)]$. The decision maker compares these results with the "ideal" vector $\overline{Z} = [M_1, M_2,\ldots,M_n]$. If the values of some components of $\overline{Z}_0$ are satisfactory and others are not, the decision maker must accept a certain reduction in the value of one or more satisfactory objectives in order to improve the unsatisfactory ones in the next iteration. The problem (7.14) is again solved with the revised constraint set. The procedure continues until the decision

maker is satisfied with the results or no suitable compromise is obtained. The concepts involved in the STEM procedure can be illustrated by means of the following examples.

Consider the problem of allocation of capacities between two purposes as discussed in Sections 3.1 and 7.2. If the STEM procedure is used, the "ideal" solution is as shown in Figure 7-3. This solution is obtained by first maximizing dry-weather BOD removal assuming all resources are devoted to this purpose, yielding M_1, and then maximizing wet-weather BOD removal assuming all resources are allocated to this purpose, yielding M_2. Note first that this solution ignores the status quo point, thereby considering the entire curve PQ as the noninferior set. If the status quo point is considered, the noninferior set would be curve P'Q'. Secondly, the subsequent decision making process requires each purpose to specify how much of the BOD removal it is willing to sacrifice in order to obtain more of the other purpose.

Now consider the case of cost sharing between two cities, 1 and 2, discussed in the previous section. The "ideal" solution using STEM is (0,0) as shown by point O in Figure 7-4. Again, it does not consider the status quo point. The existence of the status-quo point changes the "ideal" solution to point S. In this case, the decision making process requires one city to specify how much

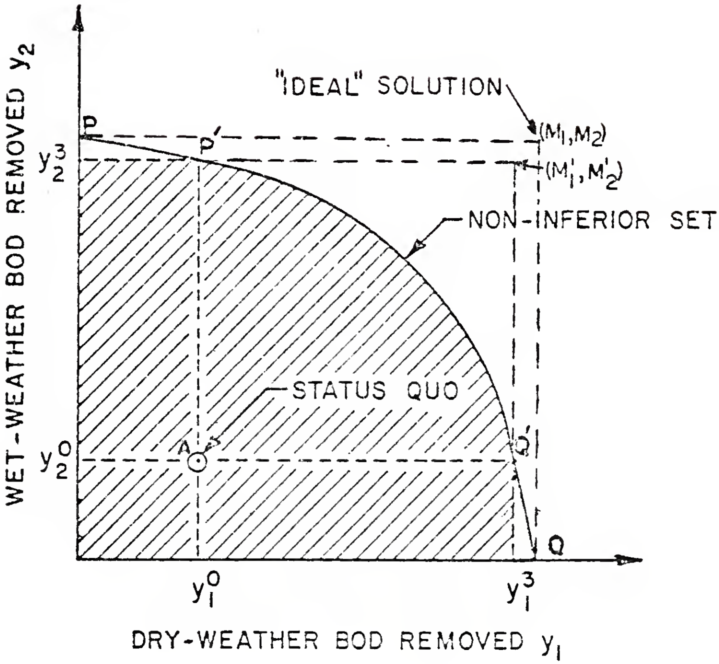


Figure 7-3. Illustration of "Ideal" Solution by STEM for Resource Allocation Problems

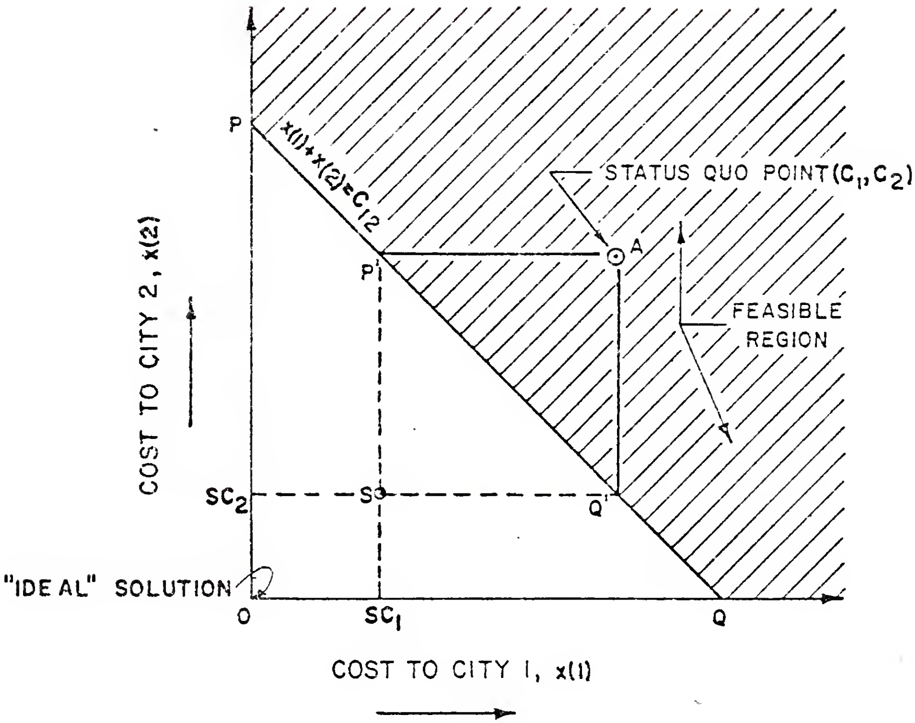


Figure 7-4. Illustration of "Ideal" Solution System for Cost-Sharing Problem

additional costs it is willing to incur in order to decrease the costs to the other city.

Haith and Loucks (1973) demonstrated the use of STEM procedure in solving the treatment plant problem for two municipalities, and a review of the example shows that the decision-making process requires one municipality to increase its degree of treatment while decreasing the other municipality.

In urban wastewater management involving allocation of a resource such as the assimilative capacity of a receiving water, treatment plant capacity, etc., each participant is usually guaranteed a certain allocation in the form of "ownership." The objective of each participant in a coordinated strategy is to increase this allocation to himself. Similarly, in cost sharing/cost allocation, each participant usually has an alternative or a set of alternatives which determines his maximum cost. Therefore, the objective of each participant in a coordinated solution is to decrease his cost. These objectives can only be attained if the participants cooperate. But the manner in which the problem is solved by the STEM procedure poses a competitive, rather than a cooperative, situation. It ignores the existence of the status quo point. Further, the "ideal" solution is an unattainable quantity and does not have meaning. Application of concepts

from cooperative N-person game theory, to be discussed later in this chapter, allows the multiobjective problems to be formulated as cooperative problems rather than competitive situations.

Among other iterative solution techniques leading to a unique solution are the iterative weighting method and iterative goal programming. In the first case, the best compromise solution is obtained by using a set of weights derived by solving a two-person zero sum game. Its implications for decision making are similar to those for STEM. Iterative goal programming requires defining "goals" or aspiration levels. Monarchi (1972) demonstrates application of this method to a water quality example. It does not appear that this method is suitable for resource allocation or cost sharing/cost allocation. The next section discusses how β_i is determined by conventional cost-sharing/cost-allocation techniques.

7.4 Conventional Cost-Sharing/Cost-Allocation Techniques

There is a considerable amount of literature related to apportioning costs (Ransmeier, 1942; Regan, 1964; James and Lee, 1971; Marshall and Broussalian, 1972; Giglio and Wrightington, 1972). Marshall and Broussalian discuss cost-allocation methods most frequently used by various federal agencies in the area of water resources development

while Giglio and Wrightington discuss methods for apportioning the costs of joint wastewater treatment plants among users. All methods have one thing in common, i.e., each purpose or group is assigned all the costs that can be unambiguously attributed to that purpose or group (separable costs) plus a portion of the joint or non-separable costs. They differ only in how the separable costs are defined and the allocation vehicle, β_i, which is used for assigning a portion of the joint costs.

Separable costs of a purpose are usually defined as the incremental costs of including that purpose in the multipurpose project, N, as in equation (7.2). However, in wastewater management, separable costs to purpose or group i are usually regarded as the costs of those project elements such as pipelines used exclusively by i. These costs are not necessarily equal to incremental costs. Thus,

$$SC_i \begin{cases} = C(N) - C(N - \{i\}), \text{ or} \\ = \text{cost of facilities used by i alone.} \end{cases} \tag{7.15}$$

James and Lee (1971) list six principal vehicles that have been used for allocating joint costs. These vehicles and the associated value of β_i are as follows:

(1) equally among centers, $\beta_i = \frac{1}{n}$; (7.16)

(2) proportionally to the quantity of use the center makes of the facilities as expressed in units such as volume, flow rate, BOD, etc.

$$\beta_i = \frac{D_i}{\sum_i D_i} \qquad (7.17)$$

where D_i = units from center i;

(3) entirely to the highest priority cost center within the limit of the benefit the center receives

$$\beta_i = \min\left[1 - \sum_{i=1}^{n-1} \beta_i ; \frac{C_i - SC_i}{NSC}\right] \qquad (7.18)$$

where C_i = cost of independent action;

(4) proportionally to the excess cost required to provide the service by some alternative means, or

$$\beta_i = \frac{C_i - SC_i}{\sum_i C_i - \sum_i SC_i} \qquad (7.19)$$

where C_i = cost of providing service to center i by some alternative means,

$\sum_i C_i$ = sum of the costs of serving all centers by alternative means.

This method may be denoted as the alternative cost method;

(5) proportionally to the benefit in excess of assigned separable cost by the given cost center. In this case, β_i is given by equation

(7.19) with C_i representing the gross benefits to center i and $\sum_i C_i$ as the total project gross benefits; or

(6) proportionally to the smaller of the excess benefits or the excess cost of the alternative project. In this case β_i is also given by equation (7.19) with C_i representing the minimum of the alternative cost and gross benefit to center i and $\sum_i C_i$ representing the minimum of the total project alternative cost and total project gross benefits.

The above methods can be illustrated graphically by means of the two city examples. In Figure 7-5, total costs (C_{12}) are represented by line PQ and the marginal costs are denoted by point S. In accordance with method 1, $\beta_1 = \frac{NN'}{2}$ and $\beta_2 = \frac{MM'}{2}$. In accordance with the second method

$$\beta_1 = \frac{NN'(D_1)}{D_1 + D_2} \quad \text{and} \quad \beta_2 = \frac{MM'(D_2)}{D_1 + D_2}$$

In accordance with the third method, the cost-sharing point is located somewhere along the line PQ.

In methods 4, 5, and 6, a point A, as shown in Figure 7-5, is defined denoting alternative costs, gross benefits, and minimum of the gross benefits or alternative costs respectively. The point of intersection, R, of the line joining points A and S with the line PQ yields the cost-sharing point.

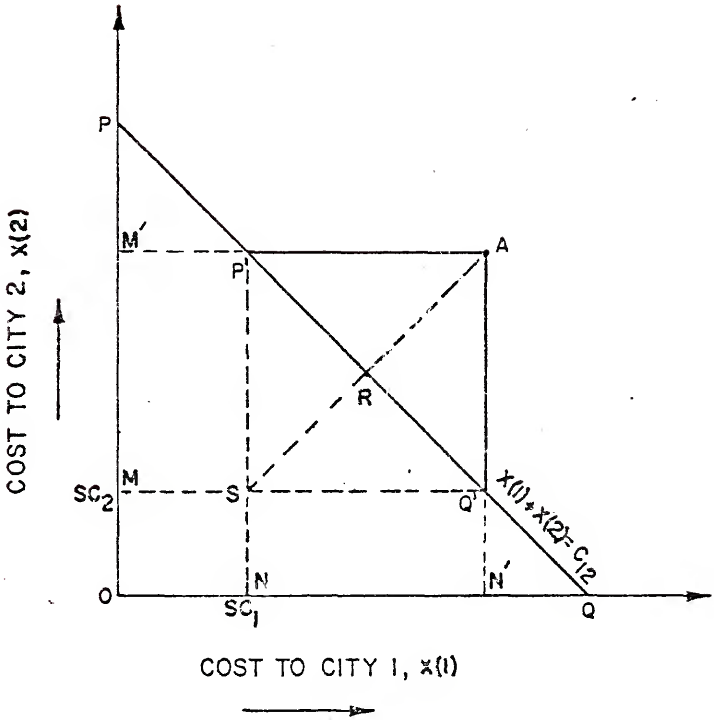


Figure 7-5. Illustration of Conventional Cost-Sharing/Cost-Allocation Techniques

In methods 1 and 2, there is no guarantee that equity criteria outlined in Section 7.2 will be satisfied. Further, method 2, also called the Use of Facility Method, is difficult to use in multipurpose problems. For example, wet-weather quantity control usage is expressed in terms of volume, dry-weather control involves flow or BOD and the wet-weather quality control usage is expressed in terms of both flow and volume. Therefore it is difficult to use a single parameter as an allocation vehicle. Since, in urban wastewater management, benefits cannot be quantified in monetary terms, methods 5 and 6 are difficult to use. Method 4 seems to meet the efficiency/equity criteria provided that the alternative costs are properly defined.

Thus, it is seen that, except for the alternative-cost method, conventional cost-sharing/cost-allocation techniques are either not equitable or unsuitable for application to urban wastewater management. The alternative-cost method is suitable only if these costs are defined properly. In the next section, a brief discussion of cooperative N-person game theory is presented. These concepts allow the cost-sharing/cost-allocation problem to be formulated as a cooperative bargaining problem rather than a competitive problem solved by STEM procedure. These concepts also assist in defining alternative costs in a proper manner.

7.5 Cooperative N-person Game Theory

Game theory is concerned with decision making situations involving conflicts and/or cooperation among

two or more decision makers (players). In general, the decision makers have different goals or preferences. The payoff received by a particular player depends not only on his decision choice, but also on those of other players. Game theory attempts to provide a normative guide to rational behavior for a group whose members aim for different goals. The game theoretic approach to collective action stems directly from "The Theory of Games and Economic Behavior" by Von Neumann and Morgenstern (1947). A general introduction can be found in Luce and Raiffa (1957), Rapoport (1950, 1966, 1970a, 1970b) and Shubik (1964a).

Game theory divides into two fundamental categories, two person and more than two person (N-person). Two-person games are further subdivided into constant sum games (also zero sum) and nonconstant sum games. In constant sum games, the interests of the players are diametrically opposed while in nonconstant sum games the interests are partially opposed and partially coincident. Nonconstant sum games and N-person games are further differentiated according to whether players can agree on joint strategies (cooperative games) or noncooperative games. Cooperative games involve coalition formation among players and provision as to how gains or losses that accrue jointly to the members of the coalition shall be apportioned among them. These provisions can be viewed as results of bargaining among the members of the respective coalitions. Thus, game theory, to the extent

that it deals with games in which coalitions can form, includes also a theory of bargaining.

Games have been studied in three forms

(1) the extensive form;

(2) the strategic or normal form; and

(3) the characteristic function form.

The extensive form of a game can be illustrated by means of a diagram known as the game tree. The game tree is determined by the rules of the game. The origin of this tree can be represented by a point. The branches issuing from the origin represent the alternatives open to the player who moves first. The end points of these "first order" branches represent the several situations which can result from the choices made by the first player. From these points, in turn, branches issue, which represent the alternatives open to the player who is to move next. The process continues until a situation is reached which is defined as the outcome of the game. An example of a game in the extensive form is the tree structure of chess. The sizes of the trees in even very simple games make mathematical analysis of games in extensive form almost impossible.

A game in normal form is simply a list of pure strategies for each one of the players along with a description of the resulting payoffs to the players for any

possible strategy choices. The theory of games in normal form is most useful in the investigation of the two-person game. In that case, the matrix is a rectangular array in which the strategies of one player are represented by rows and those of the other by columns. Zero-sum two-person games are historically the most important type of game in normal form. Of the two-person, zero-sum games, the simplest are those which have so-called saddle points in the game matrix. A saddle point is a "box" in the matrix in which the payoff to the row is minimal in its row and at the same time maximal in its column. Strategies which contain the saddle point are called maximin strategies. Each player tries to maximize his expected average payoff and there is a definite upper limit to the payoff set by the opponent's efforts to do the same. Each has a maximum "security level." In nonconstant sum games with two or more players, coalitions become feasible and players may, by mutual agreement, play in such a manner as to raise their payoffs above the security levels.

Both the extensive form and the normal form are not very convenient in situations involving many persons. A major factor in the multiperson cooperative games is that of coalition formation, and thus the maximum amount attainable by any potential coalition is of primary concern. Consequently, if the subject to be studied involves

negotiations, bargaining, or fair division procedures, it is important to look at the game in characteristic function form. An N-person game, (N,v), in characteristic function form consists of a set N = 1,2,.....,n of N players along with the characteristic function, v, which assigns the real number, v($\underline{S}$), to each nonempty subset $\underline{S}$ of players. The value, v($\underline{S}$), measures the worth or power which the coalition, $\underline{S}$, can achieve when its members act together. Thus, the only "givens" of the game in characteristic function form are the payoffs which each of the possible coalitions can assure for themselves respectively. It is assumed that side payments or "bribes" among the players are possible, i.e., exchanges among the members of a coalition to equalize any inequities arising if they cooperate. Usually, it is also assumed that the payoffs are in some conservative and transferable commodity (such as money) so that it makes sense to add the payoffs and to speak of the joint payoffs accruing to each coalition. The number of entries in a characteristic function will increase rapidly with the number of players. There will be $2^n - 1$ coalitions for an N-person game. The characteristic function satisfies the following properties:

$$v(\phi) = 0 \tag{7.20}$$

$$v(\underline{S} \cup \underline{T}) \geq v(\underline{S}) + v(\underline{T}) \tag{7.21}$$

The first property states that the worth of a coalition with no members is zero. The second property states that the worth of coalition to a union of two subsets must be at least as large as the sum of the values to the subsets separately. This last property of the characteristic function is called superadditivity. If there is strict equality in the last expression, then there is no inducement for any of the players to join in a coalition with any of the others. Such games are called <u>inessential</u>. The remaining games are called <u>essential</u>.

Given the characteristic function of a particular game and assuming that the players are rational, it is necessary to determine which of the possible coalitions can be expected to form and what the final disbursement of payoffs among players will be.

Let $x(i)$ denote the payoff to the player i if a cooperative arrangement is adopted and $v(N)$ denote the total payoff resulting from the cooperative arrangement. Then the solutions which are admissible satisfy the following:

$$x(i) \geq 0 \text{ for all } i \varepsilon N \tag{7.22}$$

$$\sum_{i \varepsilon N} x(i) = v(N) \tag{7.23}$$

The second equation expresses the group rationality condition. If we append to the above conditions, the

individual rationality condition

$$x(i) \geq v(i) \text{ for all } i \in N \tag{7.24}$$

then the solutions satisfying the above conditions are called <u>imputations</u>. In order for an imputation to be seriously considered as a potential solution to a bargaining situation, it is necessary to eliminate some of the less attractive imputations. Von Neumann and Morgenstern (1947) attack this problem by introducing the concept of <u>domination</u>. An imputation $\delta = (\delta_1, \ldots, \delta_n)$ is said to <u>dominate</u> an imputation $\gamma = (\gamma_1, \ldots, \gamma_n)$ if there exists a coalition, $\underline{S}$ (a subset of N), such that

$$\sum_{i \in \underline{S}} \gamma_i \leq v(\underline{S}) \tag{7.25}$$

$$\delta_i > \gamma_i \text{ for every } i \text{ in } \underline{S} \tag{7.26}$$

An imputation which is dominated via some set of players, $\underline{S}$, does not seem very acceptable as a solution since these players can do better by forming a coalition and "going it alone." The essential idea behind the solution concept of Von Neumann and Morgenstern is that an imputation must have some stability properties to be acceptable as a solution. However, their solution may not be unique and may consist of a set of imputations. To further narrow the set of admissible imputations, we can extend the principle of group

rationality over all subsets of $\underline{S}$ of N

$$\sum_{i \in \underline{S}} x(i) \geq v(\underline{S}) \quad \text{for all } \underline{S} \subset N \qquad (7.27)$$

The set of all imputations or payoff vectors satisfying equations (7.22), (7.23), (7.24) and (7.27) constitute the core of the game.

Thus, the first major extension of existing procedures that is revealed using the game theoretic approach is that the notion of the core of an N-person game permits one to delimit more sharply the subset of solutions which are efficient. Since it does so by examining only those solutions which are rational for each individual or subset of individuals, these solutions are also equitable solutions.

The concept of the core can be further illustrated by means of the Edgeworth trading box (Shubik, 1959) for two traders 1 and 2 with a given quantity A and B of their product, respectively. In Figure 7-6, point 0 represents the initial point at which trader 1 has A and trader 2 has B. The indifference curves of the two traders are as shown. The contract curve CC' is the solution in the economic sense. In game theory, it is the core of the game. Any imputation on CC' is not dominated by any other imputation outside CC'. Any point lying outside is not jointly optimal. It should be noted that the core does not give a unique solution. Further in a number of games the core is

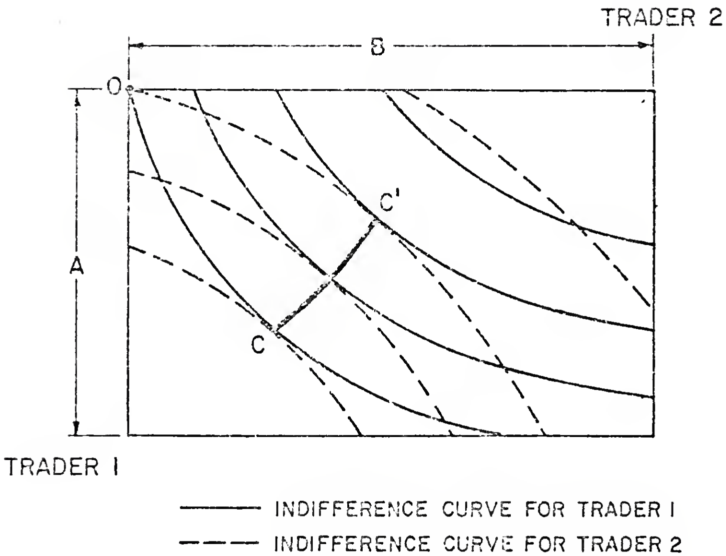


Figure 7-6. Relationship Between Edgeworth Box and Core

empty, i.e., there exists no imputation which satisfies the core conditions.

Several authors (Schmeidler, 1969; Shapley, 1953; Nash, 1970; Harsanyi, 1959; Owen, 1968; Aumann and Shapley, 1970; Raiffa, 1953) have suggested "additional assumptions" in order to obtain a unique solution in the game theoretic sense. These approaches are termed value approaches. The essential object of the value approaches is to formulate some unique valuation of the game for each participant rather than a solution such as in Von Neumann and Morgenstern's approach or as given by the core. Further, it is assumed throughout that these valuations, one for each participant, will together consistute an imputation for the game so that we are seeking some kind of a reasonable division of the maximum coalition payoff among the participants. A value is usually arrived at via various arguments arising from bargaining or arbitration schemes, from equity, reasonableness or fair division concepts or from probabilistic considerations.

Two well-known approaches are by Shapley and Nash. Each of these approaches start by defining a status quo point. Shapley defines the status quo point to represent the security levels of the players, i.e., what each can be sure to get in the game. In the Nash procedure, the status quo point consists of a choice of so-called "threat strategies" or "best" strategies available to the players.

Thus, the bargaining leverage is reflected in the Nash procedure. Luce and Raiffa (1957) call Shapley's approach arbitration and Nash's bargaining. Shapley's approach has several equity properties (1) a player's payoff is based on his incremental contribution and not on the incremental contribution of the others; (2) the sum of the individual payoffs equals the total payoff; and (3) the payoff is independent of labeling or ordering of the players. The approach suggested by Shapley is as follows:

The contribution which a player can make by joining a coalition appears,on intuitive reasons, to be an essential element of his bargaining power. Shapley considers the concept of value in an extremely general sense and postulates three basic conditions which such concepts ought to satisfy. He then shows that these conditions, in fact, determine a unique value for each participant in the game. Shapley's conditions can be viewed as general criteria of "fairness" which can be useful in an arbitration procedure. He assumes that the value of the game to each participant should not depend on the labeling of the participants and that the value for each participant should be that division of the total payoff which corresponds to his expected marginal contribution to any coalitions, whose marginal contribution is given by

$$(v[\underline{S}] - v[\underline{S} - \{i\}]) \tag{7.28}$$

If we designate by $\alpha_i(\underline{S})$ the probability that participant i will make a marginal contribution to any coalition S, then we may express the value ϕ_i of the game to participant i as

$$\phi_i = \sum_{i \varepsilon \underline{S}} \alpha_i (\underline{S}) [v(\underline{S}) - v(\underline{S} - \{i\})] \tag{7.29}$$

where the summation is over all possible coalitions among the n participants. In order to specify a magnitude for $\alpha_i(\underline{S})$, imagine a sequence in which participants join together one by one until a particular s-member coalition $\underline{S}$ is formed and that the remaining n - s players continue to join together one by one to form the complement N-$\underline{S}$ of this coalition. This gives

$$\alpha_i(\underline{S}) = \frac{(s-1)!\ (n-s)!}{n!}$$

and value ϕ_i becomes

$$\phi_i = \sum_{\underline{S} \subset N} \frac{(s-1)!\ (n-s)!}{n!} [v(\underline{S}) - v(\underline{S} - \{i\})] \tag{7.30}$$

This is known as the Shapley value for the game. Thus, if a game has a core, then the Shapley value will give a unique disbursement of payoffs which falls within the core. In many games, one or more of the group rationality conditions may be violated, in which case, the core is empty and thus

the Shapley value, although it exists, will not satisfy core constraints.

7.6 Application of Game Theory to Cost Sharing/Cost Allocation

In order to apply the above concepts to cost sharing/cost allocation, consider the urban wastewater problem involving several participants (groups or purposes). Each participant has the option of building its own separate facility or joining with one or more of the other participants in order to construct a joint facility. Assume that the coordinated solution with n participants is least costly. How should the costs of this solution be apportioned among n participants? Let

$c(i)$ = cost of the ith participant if he acts independently

$c(N)$ = total cost of the coordinated solution

$c(\underline{S})$ = total cost of the subcoalition $\underline{S}$

$x(i)$ = cost assigned to the ith participant

The core of the game is

$$x(i) \leq c(i) \text{ for all } i \in N$$

$$\sum_{i \in N} x(i) = c(N) \tag{7.31}$$

$$\sum_{i \in \underline{S}} x(i) \leq c(\underline{S}) \text{ for all } \underline{S} \subseteq N$$

To illustrate the above, consider the cost-sharing example discussed in Section 7.2. The characteristic functions of this game are

$$c(1) = C_1$$

$$c(2) = C_2$$

$$c(12) = C_{12}$$

The core of this game is

$$x(1) \leq c(1)$$

$$x(2) \leq c(2)$$

$$x(1) + x(2) = c(12) \tag{7.32}$$

The above problem can be formulated as a linear programming problem as follows:

$$\text{minimize } = x(1) + x(2) \tag{7.33}$$

subject to

$$x(1) \leq c(1)$$

$$x(2) \leq c(2)$$

$$x(1) + x(2) \geq C(12)$$

The constraint set of the feasible region is shown in Figure 7-7. The solution lies on the line segment P'Q' which is also the core of this game. It may also be viewed as the negotiation set. Note that the status quo point,

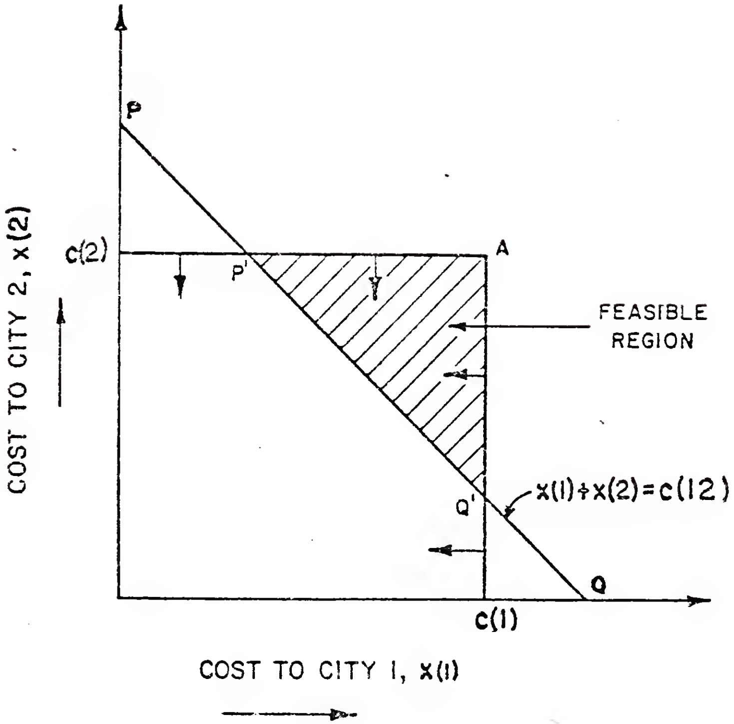


Figure 7-7. Illustrations of Game Theoretic Approach for Cost Sharing/Cost Allocation

in the game theoretical sense, is defined by the cost of each participant acting independently. The negotiation set also satisfies the following constraints:

$$x(1) \geq SC_1$$

$$x(2) \geq SC_2 . \tag{7.34}$$

The above constraints ensure that each participant pays at least its marginal cost. For $n > 2$, constraints of the above nature may further narrow the negotiation set.

Given this negotiation set, any allocation vehicle, acceptable to all participants, may be used to determine the unique solution. This solution will satisfy the efficiency/equity criteria. If the Shapley value is used as a distribution device, then

$$\phi_1 = 1/2(c(1)) + 1/2(c(12) - c(2))$$

$$= 1/2(c(1)) + 1/2SC_1 \tag{7.35}$$

$$\text{and} \quad \phi_2 = 1/2(c(2)) + 1/2SC_2 . \tag{7.36}$$

The unique solution, (ϕ_1, ϕ_2), is represented by the mid-point of the negotiation set P'Q' in Figure 7-7. It is interesting to note that, in this case, Nash's procedure yields the same status quo point, as well as the same unique solution, as that given by Shapley's approach. It will be shown in Section 7.7 that Shapley values also

ensure that each participant pays at least its marginal cost.

It should be noted that solution set, as well as the unique solution provided by the game theoretic approach, relies entirely on the assumptions embedded in the development of characteristic functions. Usually in game theory it is assumed that members not in a coalition under consideration would form a directly competitive counter coalition which leads to a minimax solution of a two-person game. Sorenson (1972) generalized some of the notions of cooperative N-person game theory to the cases where alternative definitions of the characteristic functions may be used. Three definitions are listed below.

$c_1(\underline{S})$ = value to coalition if $\underline{S}$ is given preference over $N-\underline{S}$;

$c_2(\underline{S})$ = value of coalition to $\underline{S}$ if $N-\underline{S}$ is not present;

$c_3(\underline{S})$ = value of coalition to $\underline{S}$ if $N-\underline{S}$ is given preference.

The "ideal" solution derived by STEM procedure corresponds to the first definition of the characteristic function. It was shown in Section 7.3 that this did not result in

cooperative situations. The status quo point A in Figure 7-1 is an example of the characteristic function $c_2(\underline{S})$ as the ownership of the facilities is prespecified and N-$\underline{S}$ could be viewed as not present. As seen from Figure 7-1, this leads to a cooperative situation. The situation where N-$\underline{S}$ is given preference over $\underline{S}$ can be examined by referring to Figure 7-2. This definition of characteristic function leads to status quo point A which clearly results in a cooperative situation.

Indeed one can generalize that in resource allocation and cost-sharing/cost-allocation problems, coalitions are never advantageous using $c_1(\underline{S})$ and never disadvantageous using $c_3(\underline{S})$; $c_2(\underline{S})$ is relevant only in resource allocation problems where ownership of a resource may be specified. Thus, it is seen that the security level depends upon how the characteristic function is defined. For the case where ownership is not specified _a priori_, only definition 3 of the characteristic function guarantees a cooperative situation. These results can be usefully applied in multi-objective planning as well as in cost-sharing/cost-allocation. Suppose that STEM procedure is modified so that the so-called "ideal" solution is obtained by giving preference to N-$\underline{S}$ rather than $\underline{S}$. The solution will then correspond to the status quo points shown in Figures 7-3 and 7-4. It

is seen that this solution is not only feasible but also guarantees a cooperative situation. Therefore, the decision making process would involve bargaining rather than sacrificing. This enhances the chances of implementing a plan. Similarly, $c_3(\underline{S})$ presents a reasonable basis for computing alternative costs of various coalitions.

Charnes and Sorenson (1972) mention several desirable characteristics of $c_3(\underline{S})$. Some of these are

(1) The definition of $c_3(\underline{S})$ assumes that each coalition acts rationally in its own interest. Further it provides a minimum value for each coalition given that everyone acts rationally, and thus provides a basis for bargaining. The value is obtained by allowing the opposing coalition to have the first choice; this conforms to experience where first choice in a competitive situation is desirable.

(2) For a decomposable joint cost problem $c_3(\underline{S})$ is identical to $c_2(\underline{S})$ which is probably the most natural characteristic function in scarce resource allocation problems, such as water resources development.

The game theoretic approach has been applied for allocation of joint costs. Shubik (1964b) and Littlechild (1970) set up characteristic function games to describe the allocation of joint cases. Each of these authors used

a superadditive characteristic function and used the Shapley value as a distribution device. Loehman and Whinston (1970) have also demonstated allocation of joint costs using the Shapley value as a distribution device. Use of the game theoretic approach to cost-sharing/cost-allocation with the Shapley value as a distribution device can be illustrated by means of the following examples pertaining to the hypothetical planning area.

The first example pertains to multipurpose wastewater management for city 6 as discussed in Section 6.4. The three players are

(1) dry-weather quality control

(2) wet-weather quality control

(3) wet-weather quantity control.

There are 2^n-1 or 7 possible coalitions to be analyzed.

[1] [2] [3]

[12] [13] [23]

[123]

The next step in the analysis is to find the characteristic function values for the above combinations. This represents the total cost to the coalition. These values are listed in Table 6-1. The core of this game is delimited by

$$
\begin{aligned}
x(1) \qquad\qquad &\le 108 \\
x(2) \qquad &\le 51 \\
x(3) &\le 115 \\
x(1) + x(2) \qquad &\le 157 \qquad\qquad (7.37) \\
x(1) \quad + \quad x(3) &\le 223 \\
x(2) + x(3) &\le 124 \\
x(1)+x(2)+x(3) &= 230
\end{aligned}
$$

Assuming that all coalitions are equilikely, there are 3! ways of forming the grand coalition as listed below:

123 213 312

132 231 321.

Then, the Shapley value for purpose i is the expected value of the cost summed over the above coalition formation sequences or

$\phi_1 = 1/3(108) + 1/6(157\text{-}51) + 1/6(223\text{-}115) + 1/3(230\text{-}124)$

$= \$107/\text{acre}$

$\phi_2 = \$29/\text{acre}$, and

$\phi_3 = \$94/\text{acre}$.

Note that that $\sum_{i=1}^{3} \phi_i = 230$ and that the Shapley value does fall within the core of the game.

The second example pertains to the domestic wastewater management. The least-cost solution (Strategy 6, Table 4-1) requires joint use of facilities by cities 1 through 6. In this case, city 7 is assigned its separate cost. Application of the game theoretic approach to determine cost shares for city 1 through 6 requires that 2^n-1 or 63 coalitions be analyzed and characteristic function values for all these coalitions be calculated. The cost shares using the Shapley value approach are listed in Table 7-1. Also listed in Table 7-1 are the costs for the complete decentralization strategy (Strategy 1, Table 4-1). Note that the assigned costs are less than the separate plant costs.

7.7 Relationship Between Game Theoretic Approach and Conventional Cost-Sharing/Cost-Allocation Procedures

Consider the Shapley value given by equation (7.35)

$$\begin{aligned}\phi_1 &= 1/2(c(1)) + 1/2(c(12)-c(2)) \\ &= \frac{c(1) + c(12) - c(2)}{2} \\ &= [c(12)-c(2)] + \frac{[c(1)+c(2)-c(12)]}{2}.\end{aligned}$$

Multiplying and dividing the second term in the above expression by $[c(1) + c(2) - c(12)]$ yields

Table 7-1

Apportionment of Annual Costs Using Game Theoretic

Participant	Cost Sharing Using Game-Theoretic Approach with Shapley Assumptions $/year	Separate Plant Costs $/year
City 1	$ 58,000	$ 67,000
City 2	38,000	49,000
City 3	275,000	311,000
City 4	122,000	130,000
City 5	21,000	28,000
City 6	321,000	357,000
City 7	130,000	130,000
Total	$ 965,000	$1,072,000

$$\phi_1 = c(12) - c(2) + \frac{[c(1)+c(2)-c(12)][c(1)+c(2)-c(12)]}{2c(1) + 2c(2) - 2c(12)}$$

$$= c(12) - c(2) + \frac{[c(1)-(c(12)-c(2)][c(12)-(c(12)-c(2) + c(12)-c(1))]}{[c(1)+c(2)] - [(c(12)-c(2))+(c(12)-c(1))]} \tag{7.38}$$

But $\quad c(12) - c(2) = SC_1$

and $\quad c(12) - c(1) = SC_2$

and $\quad c(12) - (c(12)-c(2)+c(12)-c(1)) = NSC$

Therefore, by substitution in equation (7.38)

$$\phi_1 = SC_1 + \frac{c(1) - SC_i}{\sum_{i=1}^{2} c(i) - \sum_{i=1}^{2} SC_i} \quad NSC \tag{7.39}$$

Comparing equation (7.39) with equation (7.5) yields

$$\beta_1 = \frac{c(1) - SC_1}{\sum_{i=1}^{2} c(i) - \sum_{i=1}^{2} SC_i} \tag{7.40}$$

The above value of β_1 is identical to that which would be obtained from equation (7.19) if alternative cost is used as an allocation vehicle. Therefore, equation (7.39) shows that the costs assigned by the Shapley value in a two person game are identical to the alternate cost method discussed in Section 7.4. It is also seen that the Shapley

values will always be on line segment P'Q' (Figure 7-7). When $n > 2$, the cost assignment using the Shapley value may not be identical with the costs assigned by using alternative-cost approach because the Shapley value takes into account more information (additional coalitions and their probability) than does the alternative-cost method. A comparison of the number of characteristic functions used in each of the two approaches is presented in Table 7-2.

Table 7-2

Comparison of Characteristic Function Used by Alternative-Cost Method and Game-Theoretic Approach

Number of Users	Alternative Cost-Method	Game-Theoretic Approach
2	2	2
3	7	7
4	9	15
5	11	31
6	13	63

A comparison of cost apportionment using the Shapley value approach and the alternative-cost approach is presented

in Table 7-3 for the domestic wastewater management example. The results are quite close. Thus, the alternative-cost approach may be viewed as an approximation of the Shapley value approach for large n. This result is quite useful because as the number of participants in a game increases, calculation of all characteristic function values, as well as the computation of Shapley values, becomes tedious. Therefore, use of the alternative-cost approach allows simplification of the computations. The alternative-cost approach has been criticized because of the difficulties in defining proper alternatives (James and Lee, 1971). However, the game theoretic approach allows these costs to be defined unambiguously as discussed in the previous section.

7.8 Summary

This chapter has presented criteria for integrated efficiency/equity analysis. Economists have argued that for efficiency and equity, it is necessary that total costs be covered and that each user be assessed on the basis of marginal cost of serving that user. Using this argument as the criterion, the cost-sharing/cost-allocation problem involves selection of an appropriate allocation vehicle for allocating the joint or nonseparable costs. Of the conventional cost-sharing/cost-allocation procedures, because of

Table 7-3

Comparison of Cost Apportionment
Using Game Theoretic Approach vs Alternative Cost Approach

Domestic Wastewater Management—Hypothetical Planning Area

Participant	Cost Assignment		Separate Plant Cost
	Alternative Cost Method	Game Theoretic Approach with Shapley Assumptions	
City 1	$ 58,000	$ 58,000	$ 67,000
City 2	39,000	38,000	49,000
City 3	276,000	275,000	311,000
City 4	120,000	122,000	130,000
City 5	21,000	21,000	28,000
City 6	321,000	321,000	357,000
City 7	130,000	130,000	130,000
Total	$ 965,000	$ 965,000	$1,072,000

the difficulties in defining benefits, only the alternative-cost approach ensures efficiency/equity in wastewater management, provided that the alternative costs are properly defined. The concepts from N-person game theory permit us to define these costs. Further, those concepts, when applied to allocation problems, yield a negotiation set which also meets the following efficiency/equity criteria: (1) each user must pay at least its marginal cost; and (2) the cost assigned to each user is no more than his cost to use an alternative strategy. Given this negotiation set, one may utilize any allocation vehicle, acceptable to all the particpants, for allocating the joint costs. Such a vehicle could be the alternative-cost approach, game theory value approaches or even the environmental and social impacts, if these can be quantified in common terms.

In problems involving a large number of participants, it may be cumbersome to check whether the solution satisfies the core conditions as well as the marginal cost constraints. In such situations, it is suggested that the solution be checked only for a few, but relatively important, constraints. In cases where the grand coalition is comprised of one more more inessential subcoalitions, the Shapley values may not fall in the core. In such cases it is necessary to check the subcoalitions in order to determine that subcoalition which offers a player the most savings.

The game theoretic approach can also be useful in multiobjective solution techniques such as STEM. The concepts allow the decision making problem to be formulated as a cooperative, rather than a competitve, situation.

CHAPTER 8

SUMMARY

This research has developed procedures for efficient and equitable urban wastewater management. The three main areas of concern in urban wastewater management which were examined were (1) control of highly polluted domestic wastewater; (2) control of quantity of stormwater runoff; and (3) control of stormwater runoff quality. In the past, efforts concentrated primarily on the first two areas. However, the recent legislation (Federal Water Pollution Control Act Amendments of 1972) has recognized the need for wastewater management in all three areas in order to achieve goals of "swimmable and fishable" waters by 1983. Consequently, Section 208 of the Act requires area-wide wastewater management in those urban areas which have severe water quality problems. The objective of the planning is the identification of an alternative for urban wastewater management that is cost effective, implementable and publically acceptable. Since an urban area usually involves several separate political entities, the urban wastewater management problem, in the context of 208 planning, is a multipurpose, multiobjective and multigroup problem.

Concepts from economic theory as well as from multiobjective planning as they relate to urban wastewater management were discussed and the importance of efficiency as well as equity was outlined. Identification of a cost-effective solution involves formulation of alternative strategies and determining their associated costs as well as environmental and social inputs. Therefore, procedures for formulating alternative strategies and determining their annual costs were described. The problem of evaluating environmental and social aspects was not addressed.

Formulation of wastewater management strategies and determination of their costs require a knowledge of wastewater flows, pollutant loading, performance data on wastewater control devices, and their associated capital and operation and maintenance costs. This information was developed in Chapter 2.

Strategies for urban domestic wastewater management can be formulated and their costs determined in accordance with the procedures presented in Chapter 4. Strategies for wet-weather quantity and quality control can be formulated in accordance with procedures presented in Chapter 5. Multipurpose wastewater planning is discussed in Chapter 6. Procedures are also presented in that chapter for determining costs associated with various multipurpose strategies.

An essential part of the implementation phase is the assignment of financial responsibility of the selected

strategy among various groups/purposes. This analysis is termed integrated efficiency/equity analysis and involves apportioning of joint costs among purposes/groups in a manner that is equitable and thus encourages implementation. Multiobjective solution techniques as well as conventional cost-sharing/cost-allocation techniques were reviewed in Chapter 7. A brief review of the N-person game theory was then presented. It was shown how these concepts are also useful in multiobjective solution techniques. The game theoretic approach was then related to conventional cost sharing techniques. The procedures discussed in this work were illustrated by means of a hypothetical planning area example.

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BIOGRAPHICAL SKETCH

Sheikh Mohammad Hasan was born on July 10, 1933 in Jullundur City, India. He received a Bachelor of Science degree in civil engineering from the Punjab University Lahore, Pakistan, in 1953. He moved to the United States in 1960 after working for a number of years with the West Pakistan Public Works Department. In September, 1961, he received a Master of Science degree in Sanitary Engineering from Northwestern University, Evanston, Illinois. He worked with various consulting firms in the U. S. prior to enrolling in the Graduate School of the University of Florida in 1972.

His publications include "Benefit-Cost Ratios of Water Supply Systems," with E. E. Pyatt and P. P. Rogers. Journal of Land Economics, 1964; "Game Theoretic Approach to Equitable Environmental Quality Management," with J. P. Heaney. NATO Conference, Istanbul, Turkey, 1973; "Methodology for Evaluating the Cost of Urban Stormwater Quality Management," with J. P. Heaney. ORSA/TIMS Joint Meeting, Chicago, Illinois, 1975.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

James P. Heaney, Chairman
Associate Professor of
Environmental Engineering
Sciences

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Wayne C. Huber
Associate Professor of
Environmental Engineering
Sciences

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Edwin E. Pyatt
Professor and Chairman of
Environmental Engineering
Sciences

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Richard L. Francis
Professor of Industrial and Systems Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Clyde F. Kiker
Assistant Professor of Food and Resource Economics

This dissertation was presented to the Graduate Faculty of the Department of Environmental Engineering and to the Graduate Council, and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy.

March, 1976

Dean, College of Engineering

Dean, Graduate School